Grid Slabs.
Panelled Beams.

نسألكم الدعاء

IF you download the Free ***APP.*** RC Structures YASSER EL LEATHY *on your smart phone or tablet, you will be able to play illustrative movies For any paragraph that has a QR code icon*

اذا حملت تطبيق RC Structures YASSER EL LEATHY على تليفونك المحمول او اللوح السطحى ستستطيع أن تشغل أفلام شرح للمقاطع التى تحتوى على رمز

Panelled Beams. Table of Contents.

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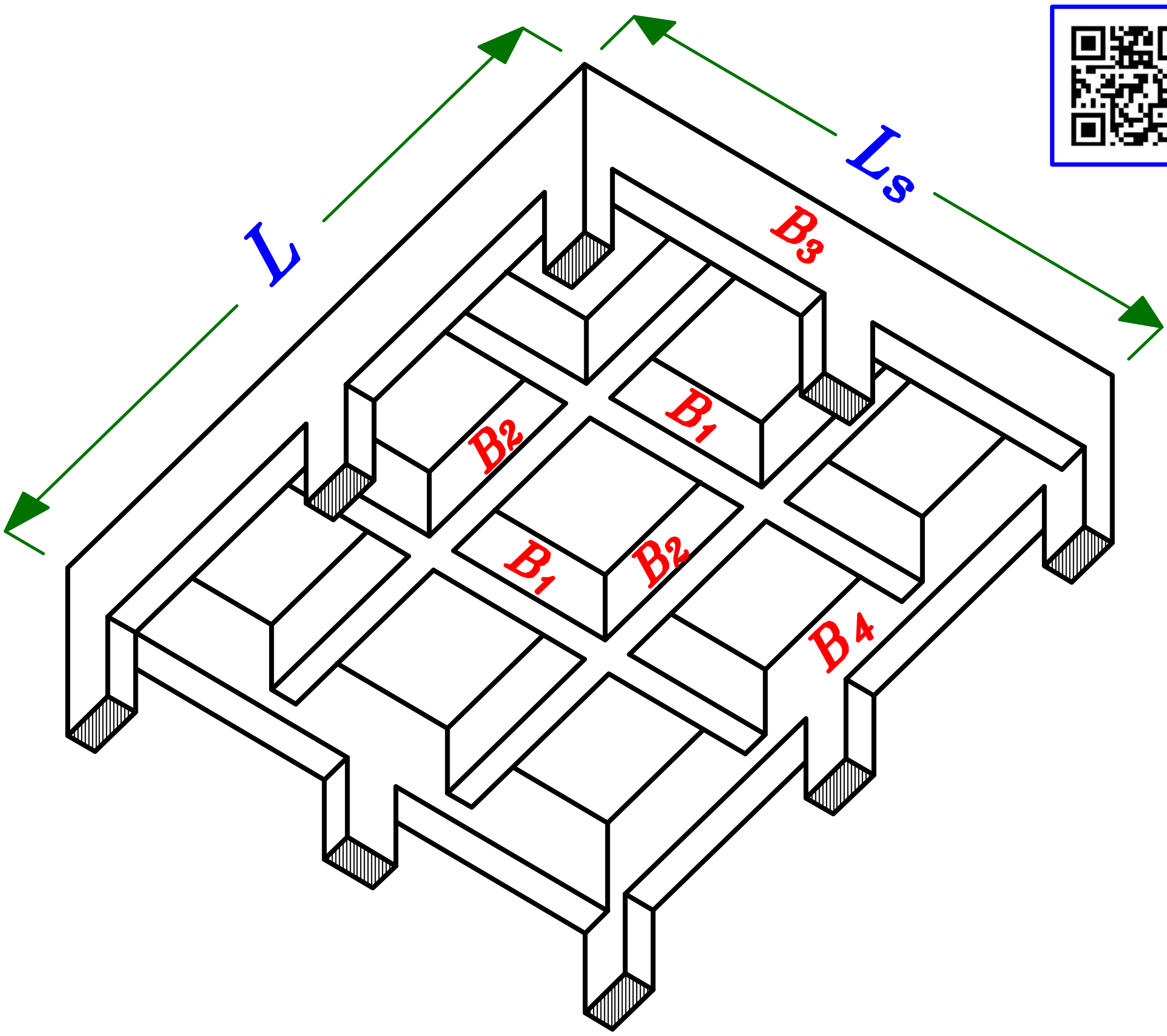
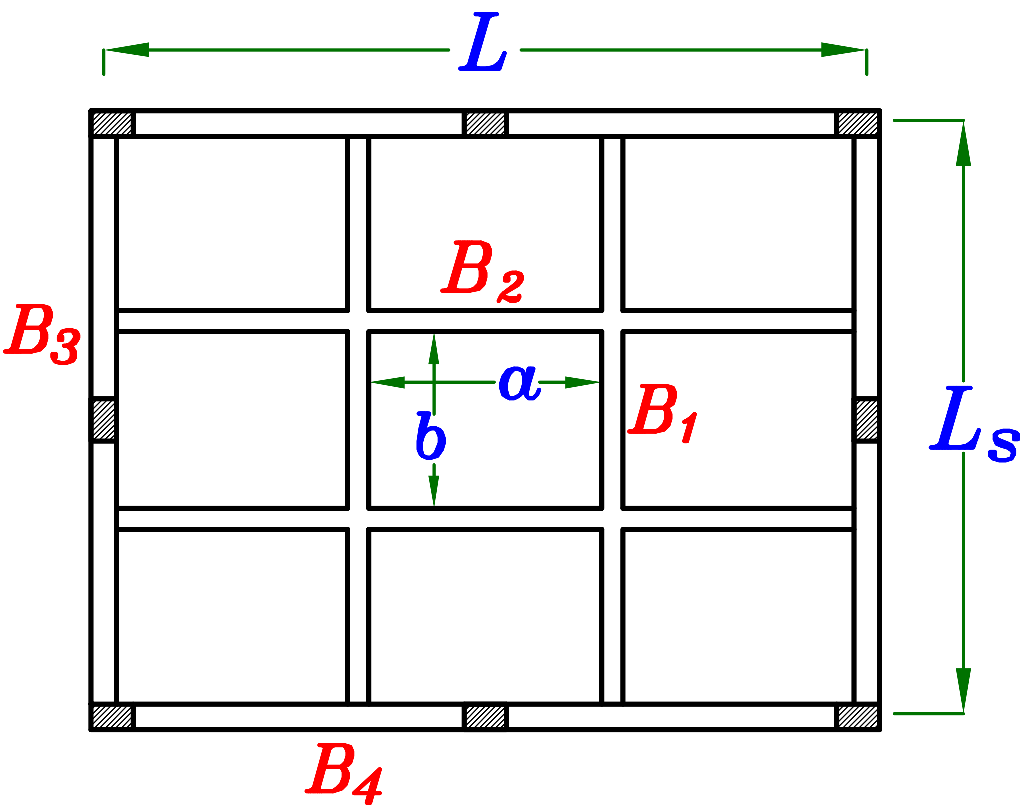
Introduction.





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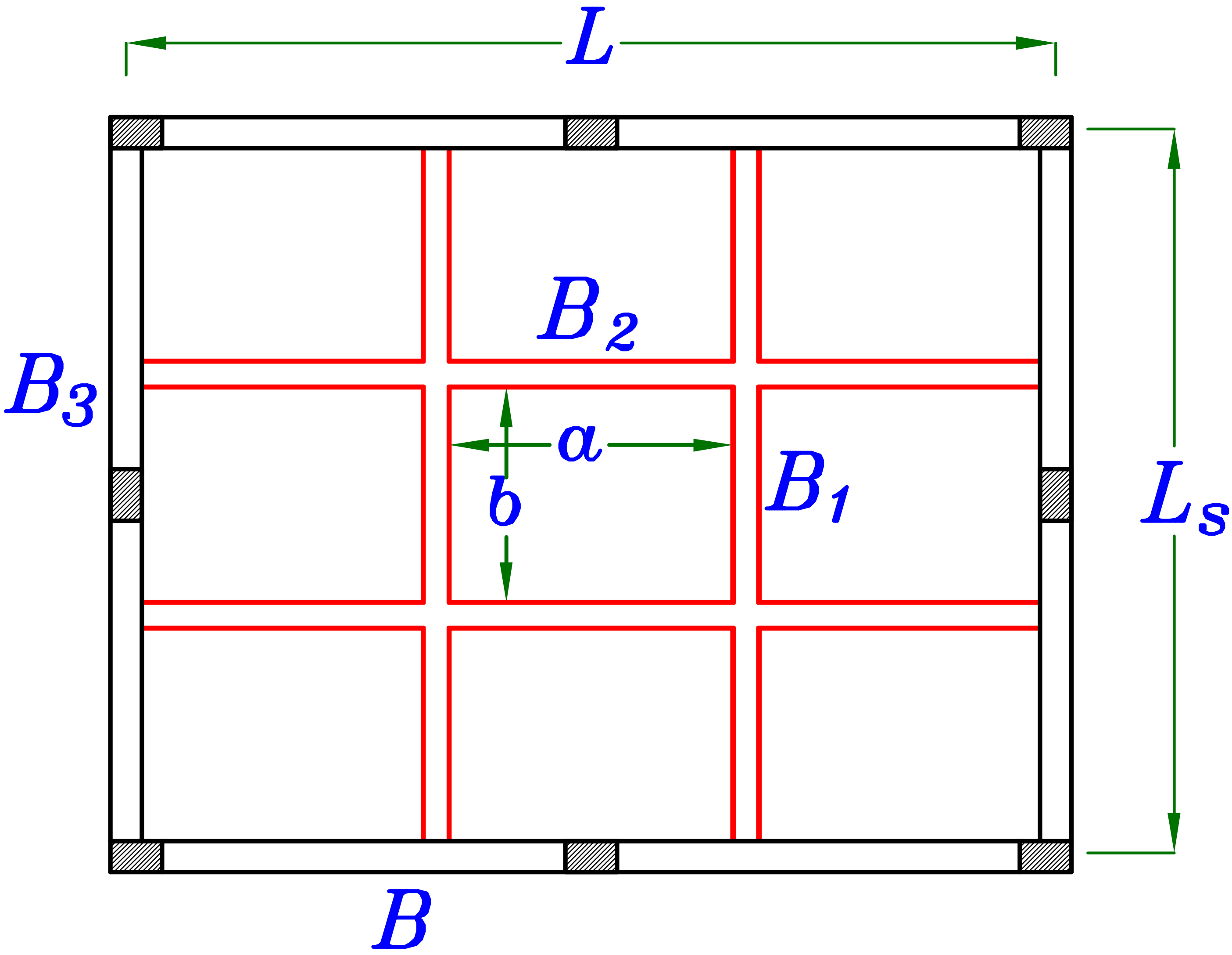

– ال Panelled Beams هى عباره عن كمرات متقاطعه B_1 , B_2 مكوّنه شبكه من الكمرات لتعمل Grid Action .

– و عند تقاطع هذه الكمرات مع بعضها تكوّن بينها بلاطات صغيره $(a * b)$ و هذه البلاطات إما أن تكون Solid Slab or Hollow Blocks

– يفضل أن تكون المسافات بين الكمرات الداخليه $(a \text{ or } b)$ من $(2.0\,m \rightarrow 5.0\,m)$.
و لا يشترط أن تكون المسافات بين الكمرات مسافات متساويه .

– تستخدم الPanelled Beams للبلاطات ذات المساحات الكبيره $(80\,m^2 \rightarrow 150\,m^2)$ بدون وضع أعمده فى الداخل و اذا زادت المساحه عن ذلك نأخذ تأثير الاعمده و نعمل Panelled Frames و فى هذه الحاله ممكن أن تصل المساحه الى $(500\,m^2)$

– ممكن وضع أعمده خارجيه لتقليل ال B.M. على الكمرات الخارجيه B_3 , B_4 .

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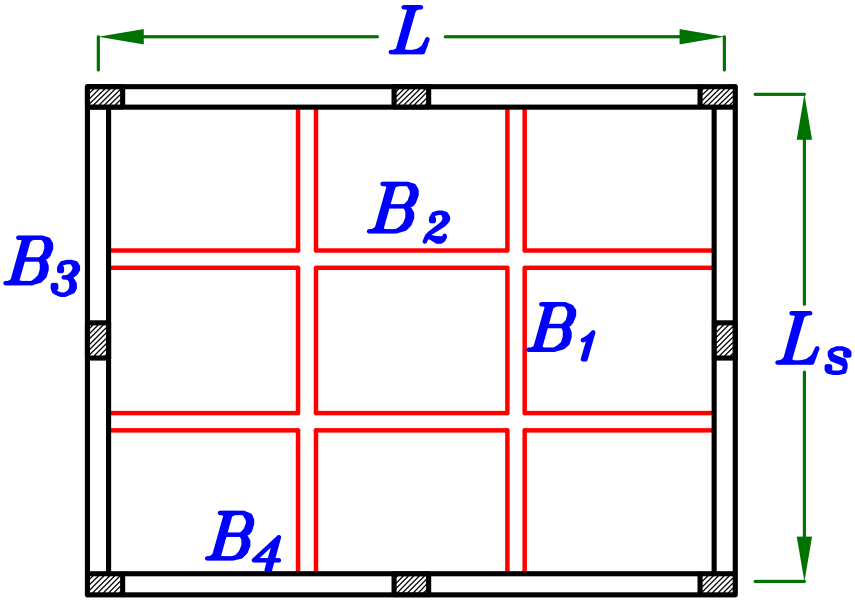


البلاطات محموله على الكمرات المتقاطعه **Panelled Beams**

و الكمرات الـ **Panelled Beams** محموله على الكمرات الخارجيه **Edge Beams**

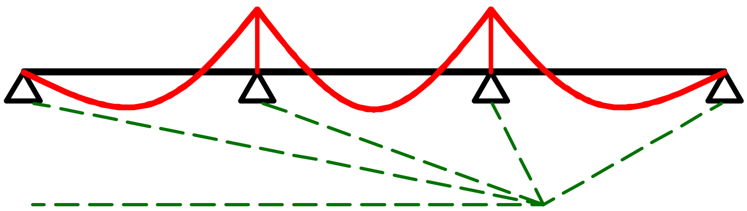
و الكمرات الـ **Edge Beams** محموله على الاعمده .

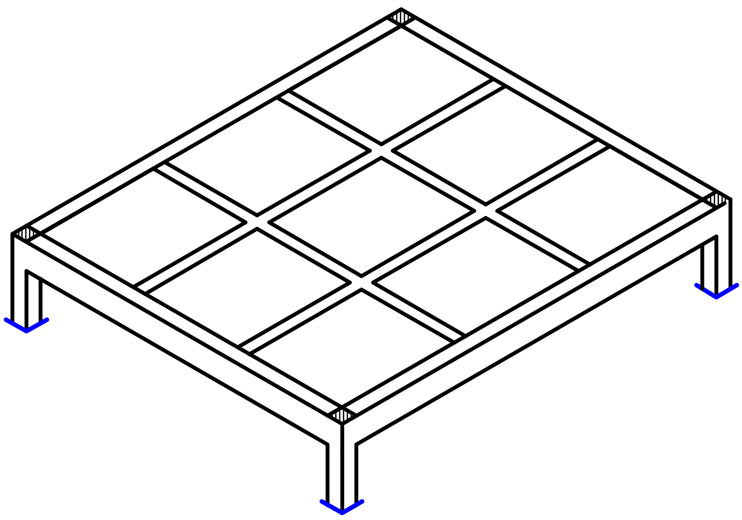
لان الـ **Stiffness** للكمرات الـ **Panelled** أكبر بكثير من البلاطه

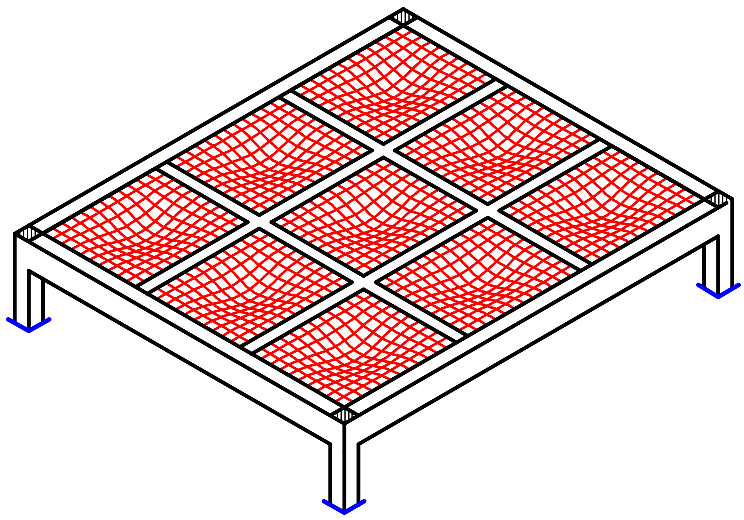
اذاً البلاطه تعتبر محموله على الـ **Panelled Beams** و الـ **Edge Beams**

اى اننا اعتبرنا الكمرات الـ **Panelled** و الـ **Edge Beams**

كأنها **Rigid Supports** للبلاطه .







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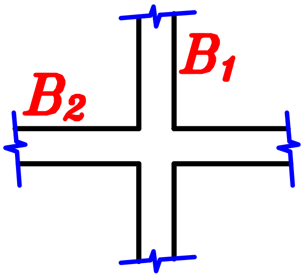
ال **Panelled Beams** تكوّن شبكه من الكمرات محموله على الكمرات الخارجيه فقط.

و الكمرات ال **Panelled** المكونه للشبكه لا يوجد أياً منها تحمل الاخرى .

أى لا يوجد كمره حامله و كمره محموله .



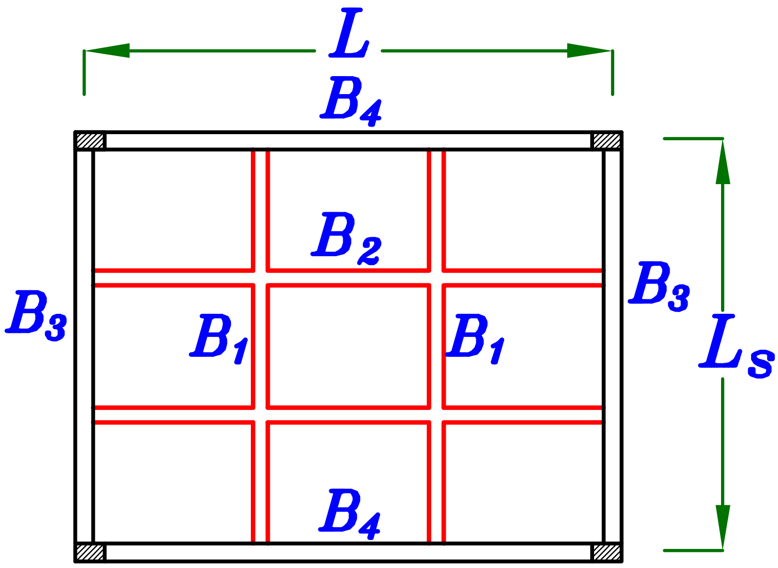
لذا عند رسم تقاطع الكمرات فى ال **Plan** ترسم كما بالشكل

و الفائده من عمل الشبكه هو انه لا توجد كمره تحمل الاخرى .

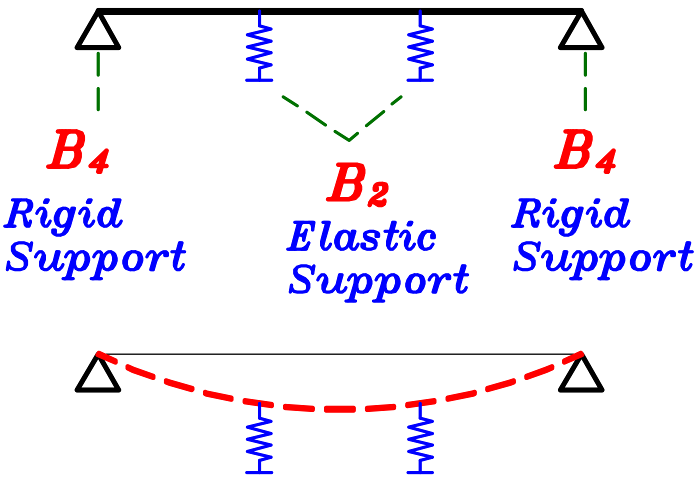
فيتوزع الحمل فى الاتجاهين فيقل ال **Deflection** و بالتالى تقل قيمه ال **moment**

و بالتالى عند التصميم تكون كميه الخرسانه و الحديد المطلوبين اقل و بالتالى تكون الكمرات أرخص .

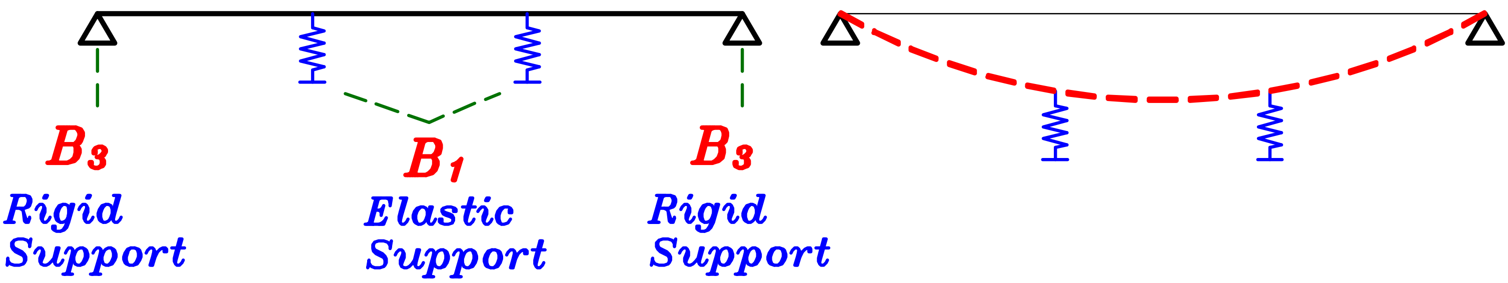
كل كمره **Panelled** تعمل بالنسبه للكمره العموديه عليها كأنها **Elastic Support**



B_1



B_2



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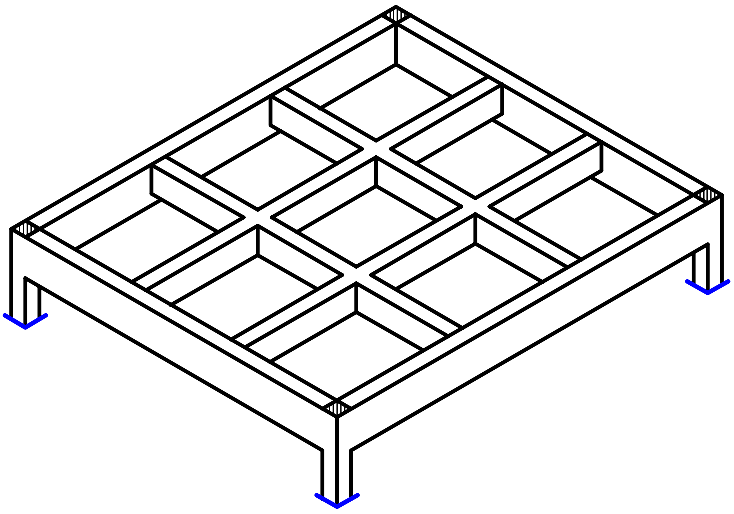
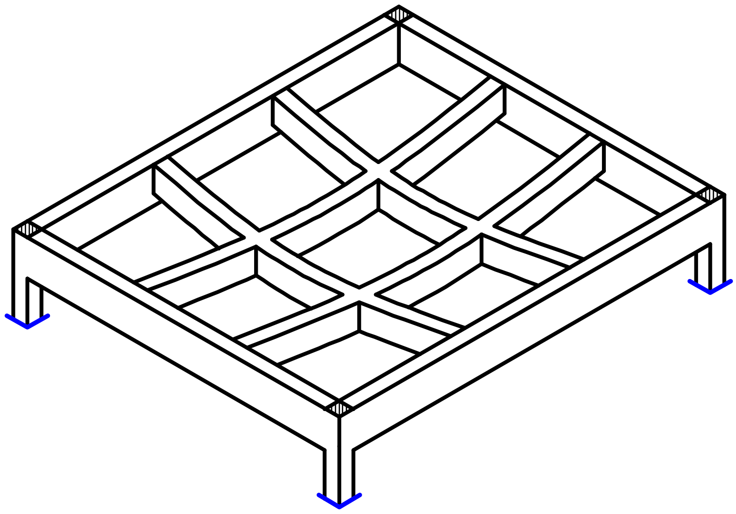
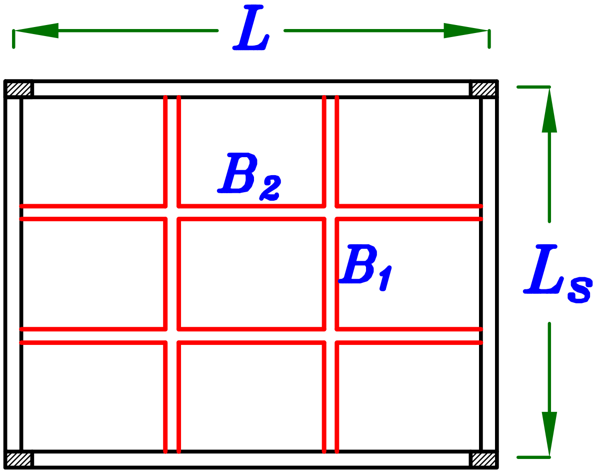


Before Loading



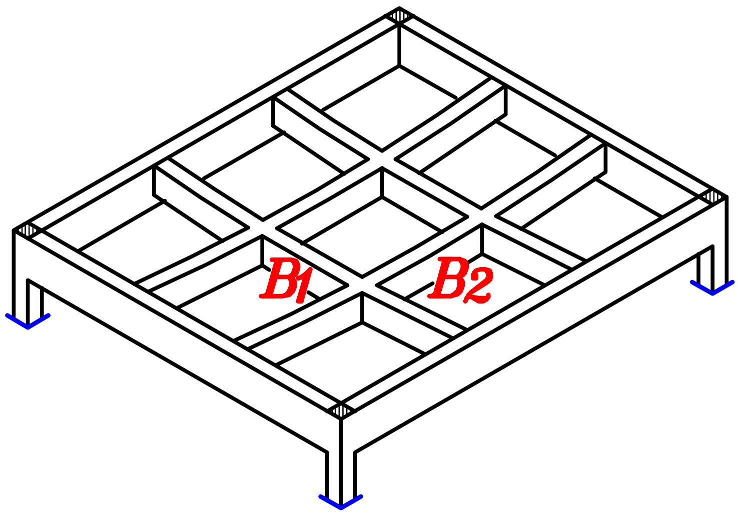
After Loading

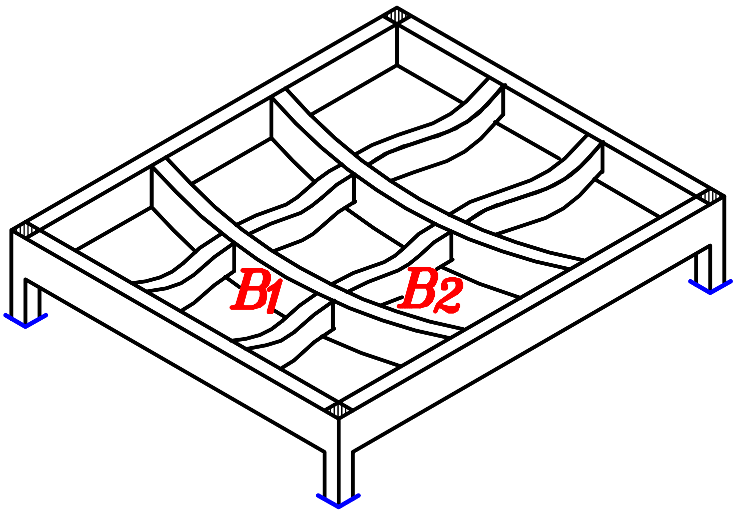
و لكى نضمن أن الكمرات الـ **Panelled** لا تحمل الكمرات الـ **Panelled** العموديه عليها .
يجب أن تكون الـ **Stiffness** لكل من الكمرتين تقريبـاً متساويه .



Stiffness

$$\left(\frac{E\,I}{L_s}\right)_{B1} \simeq \left(\frac{E\,I}{L}\right)_{B2}$$





Stiffness $B1 \simeq$ Stiffness $B2$

Panelled Beams

لا توجد كمره محموله على الاخرى من B1 و B2

Stiffness $B1 >$ Stiffness $B2$

Secondary Beams & Girders

الكمره B2 محموله على الكمره B1

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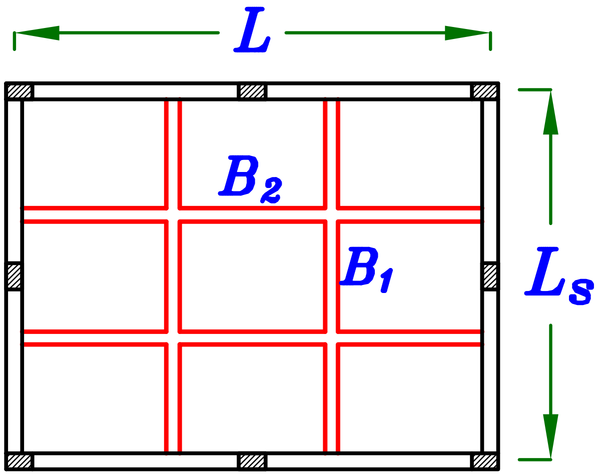


Stiffness

$$\left(\frac{E\,I}{L_s}\right)_{B1} \simeq \left(\frac{E\,I}{L}\right)_{B2}$$

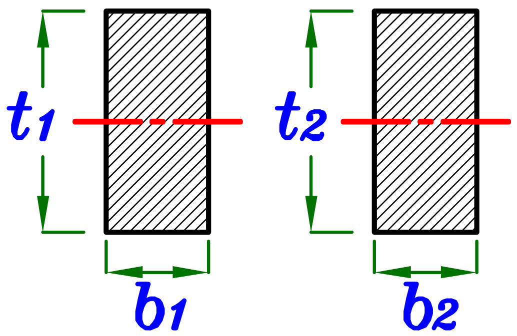
* $E \longrightarrow$ *Material of the Beam.*

أى يجب أن تكون الـ Material للكمرتين واحده (خرسانه مسلحه)

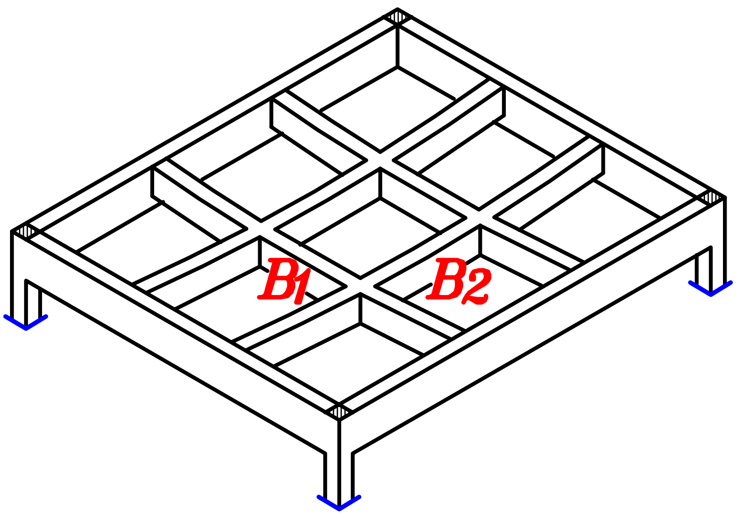
$$(E)_{B1} = (E)_{B2}$$

* $I \longrightarrow$ *Dimensions of the Section.*

أى يجب أن تكون أبعاد القطاع لكلا الكمرتين متساوى $b_1 = b_2 \;,\; t_1 = t_2$

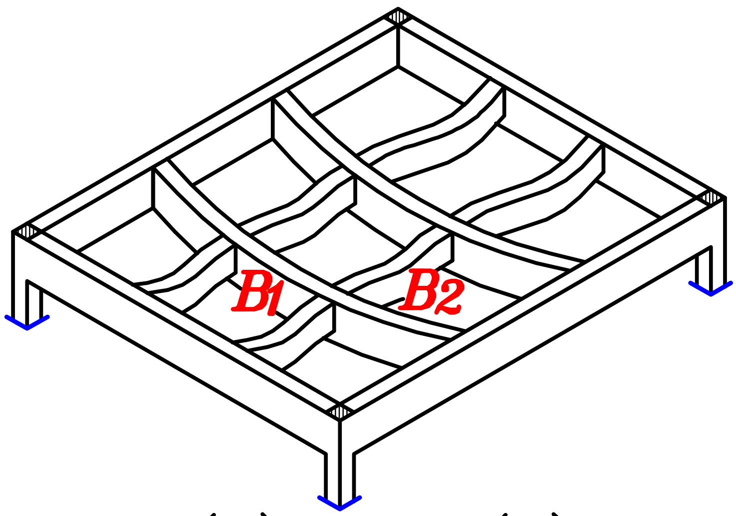


$$(I)_{B1} = (I)_{B2}$$



$$(I)_{B1} = (I)_{B2}$$

لا توجد كمره محموله على الاخرى من B_1 و B_2



$$(I)_{B1} \ggg (I)_{B2}$$

الكمره B_2 محموله على الكمره B_1

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* $L \longrightarrow$ Length of the Beam.

أى يجب أن يكون طول الكمرتين تقريباً متساوى .

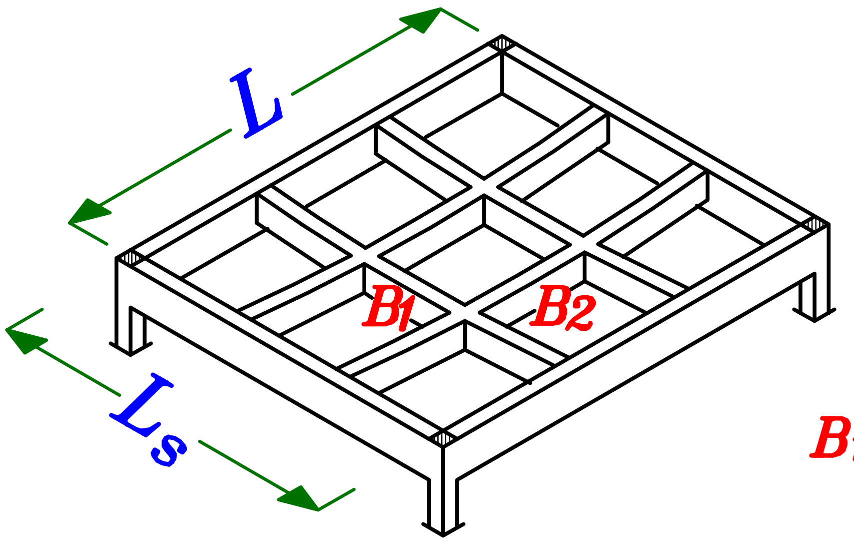
$$(L)_{B1} \simeq (L)_{B2}$$

و حتى اذا كان الطولان مختلفان $(L)_{B1} \neq (L)_{B2}$ يجب ان يكون الفرق بينهم قليل .

لذا شرط فى البلاطات ال Panelled Beams

فى الكود $\boxed{\frac{L}{L_s} \ngtr 1.5}$

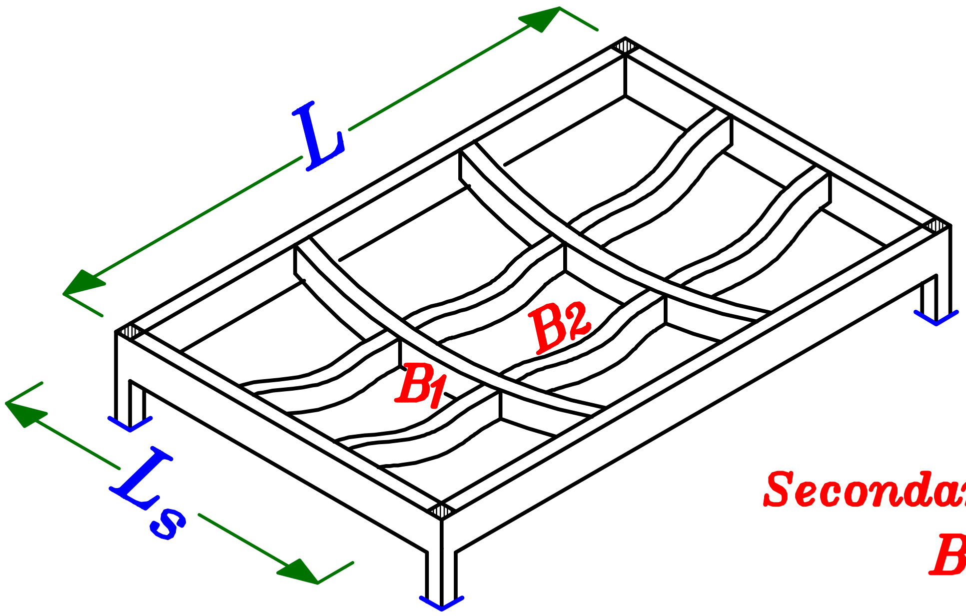
يفضل عملياً $\boxed{\frac{L}{L_s} \ngtr \frac{4}{3}}$



$\boxed{\frac{L}{L_s} \ngtr \frac{4}{3}}$

Panelled Beams

لا توجد كمره محموله على الاخرى من B1 , B2



$\boxed{\frac{L}{L_s} > \frac{4}{3}}$

Secondary Beams & Girders

الكمره B2 محموله على الكمره B1

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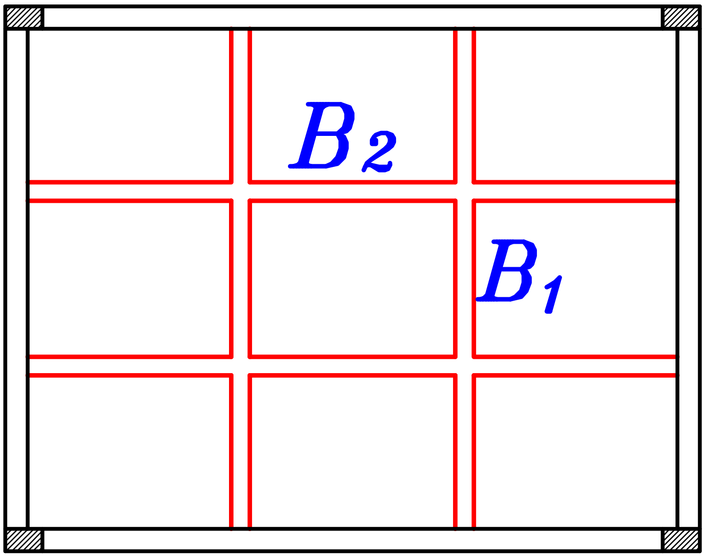
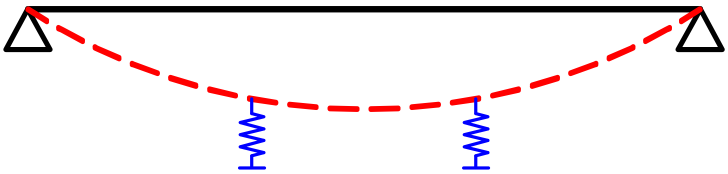
Types of Panelled Beams.

① Simple Panelled Beams.

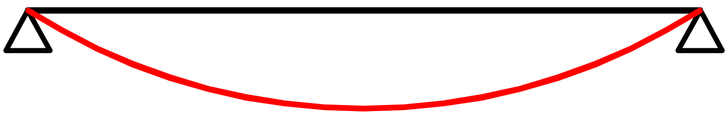
Simple & Perpendicular. Panelled Beams.



Deflection



B.M.D.



M * Reduction Factor

سيتم حساب ال Reduction Factor لاحقاً

سيتم دراسه هذا النوع فى هذا الملف .

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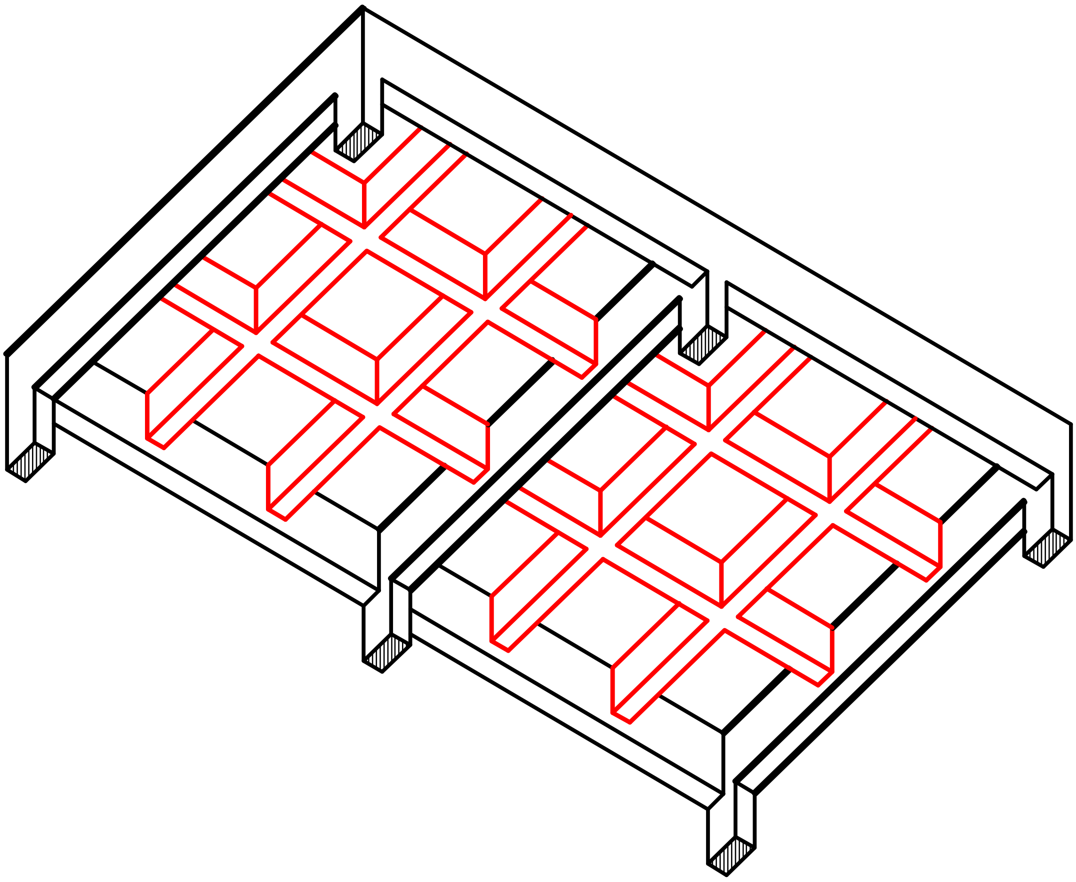
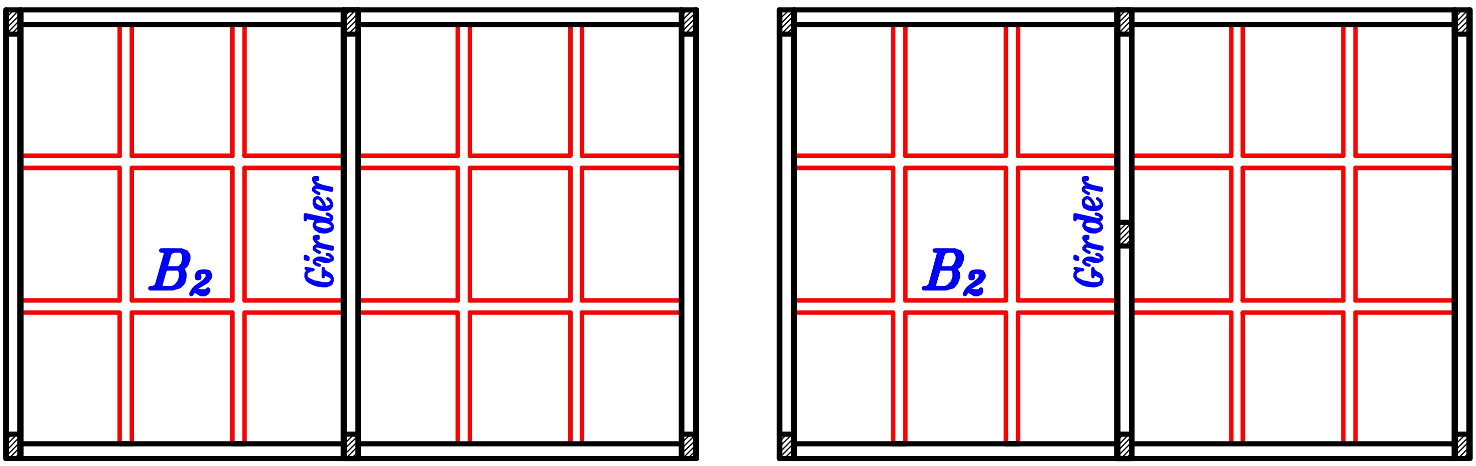
② Continuous Panelled Beams.





لكى نضمن ان ال Girder سيحمل الكمره B_2

يجب ان تكون ال Stiffness لل Girder أكبر بكثير من الكمره B_2

إما عن طريق زياده أبعاد قطاع ال Girder لزياده ال I لل Girder

او وضع عمود فى منتصف ال Girder لتقليل ال L لل Girder

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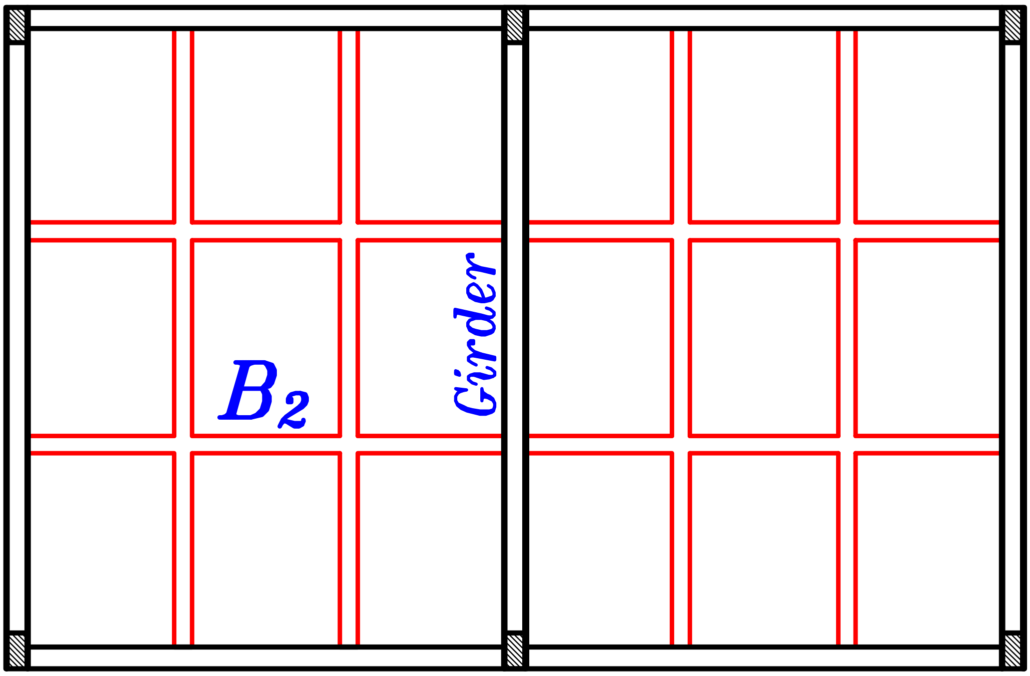
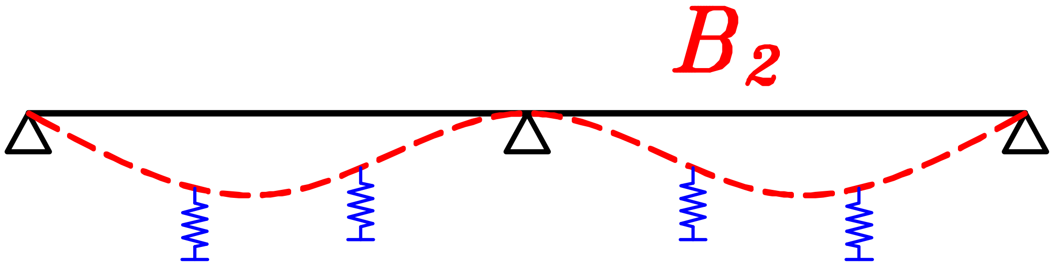
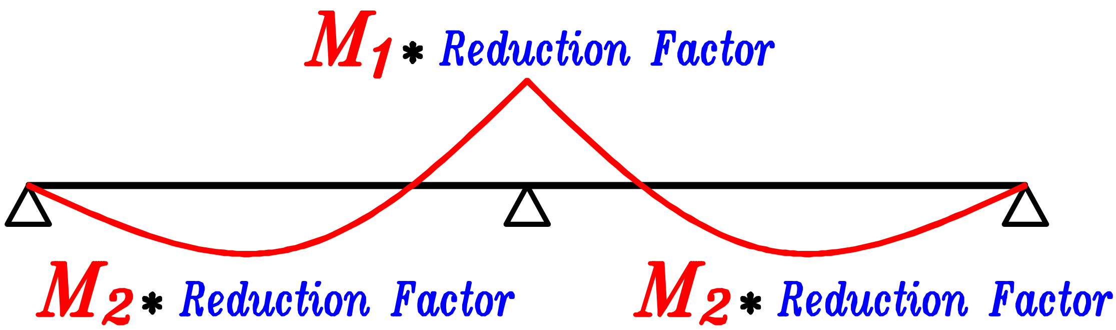


Deflection



B.M.D.



لن نستطيع حساب الـ ***Reduction Factor*** بالطرق اليدويه
لذا سنضطر لحساب الـ ***Continuous Panelled beams***
عن طريق الكمبيوتر .

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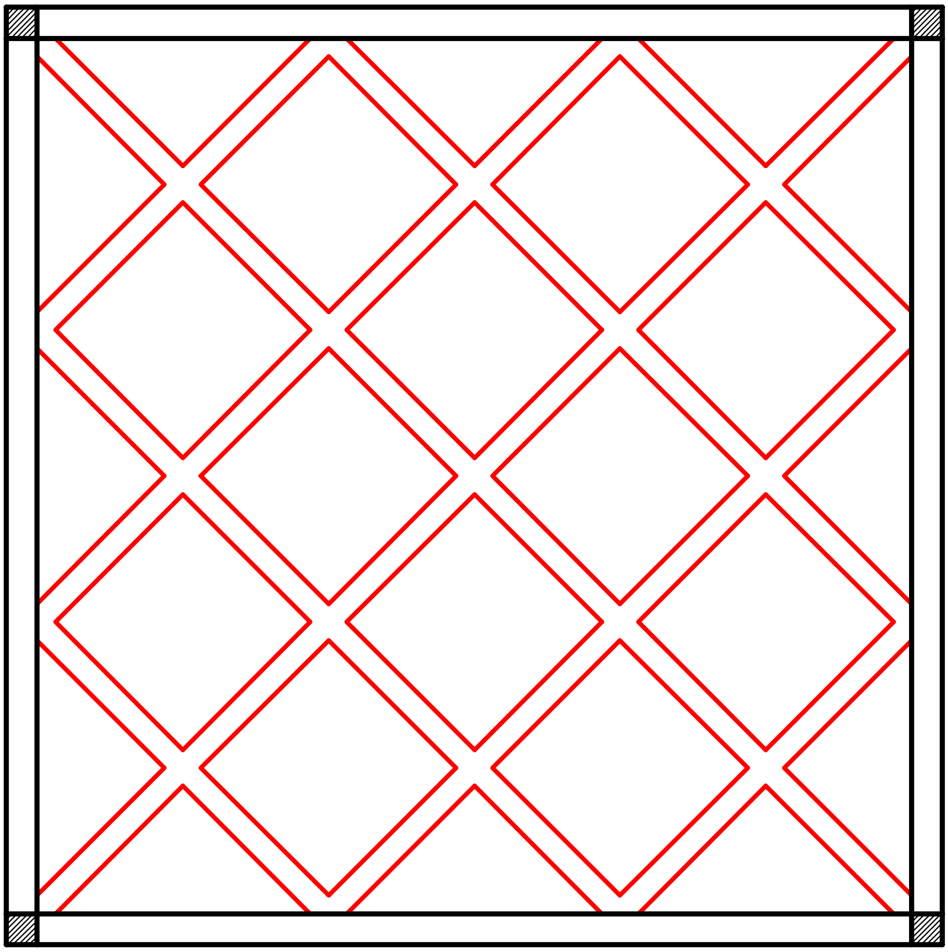
③ *Skew Panelled Beams.*



لان أطوال الكمرات مختلفه فتكون ***Stiffness*** الكمرات مختلفه و بالتالى يكون توزيع الاحمال مختلف .

لن نستطيع حساب الـ ***Reduction Factor*** بالطرق اليدويه لذا سنضطر لحساب الـ ***Skew Panelled Beams*** عن طريق الكمبيوتر .

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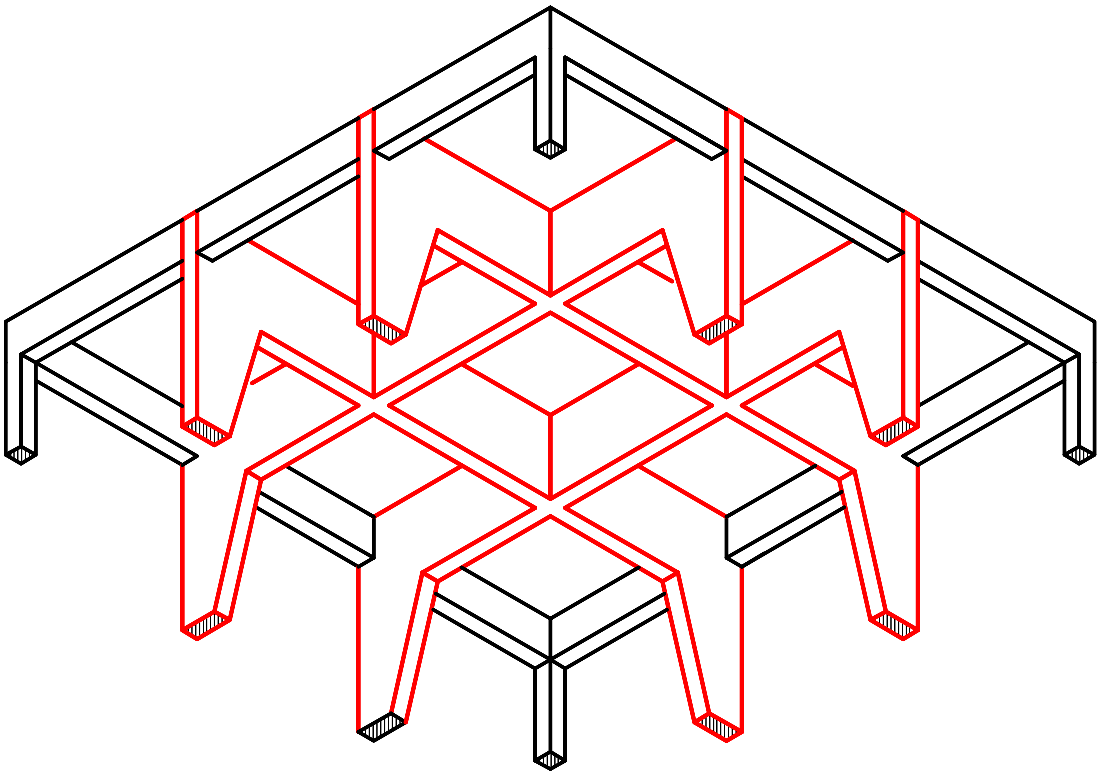
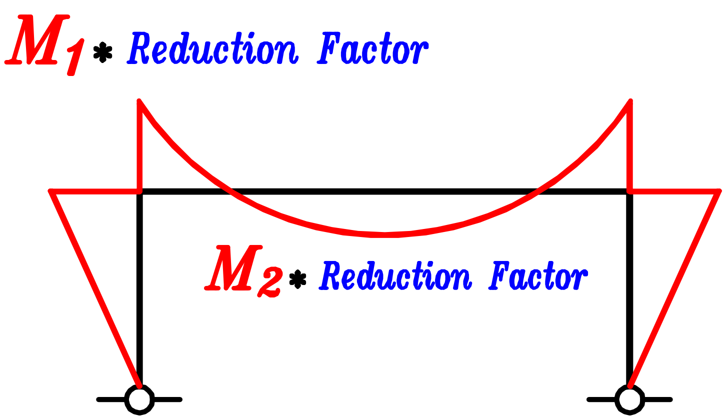
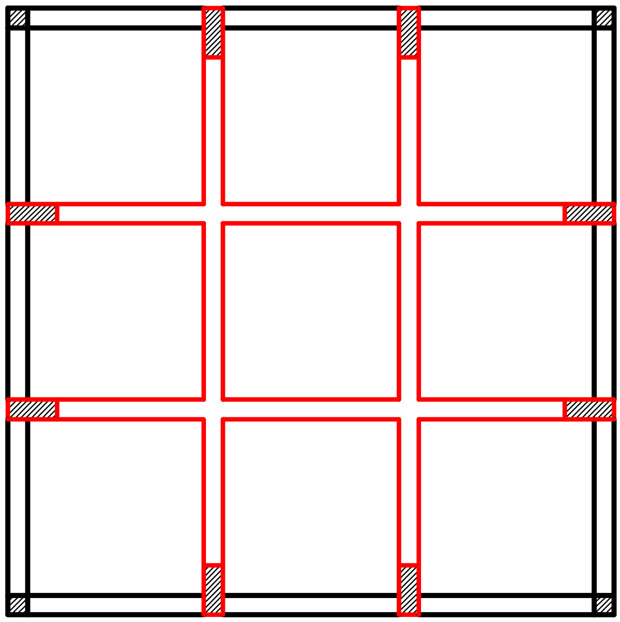
④ Panelled Frames.

Simple & Perpendicular. Panelled Frames.







لن نستطيع حساب الـ Reduction Factor بالطرق اليدويه
لذا سنضطر لحساب الـ Panelled Frames عن طريق الكمبيوتر .

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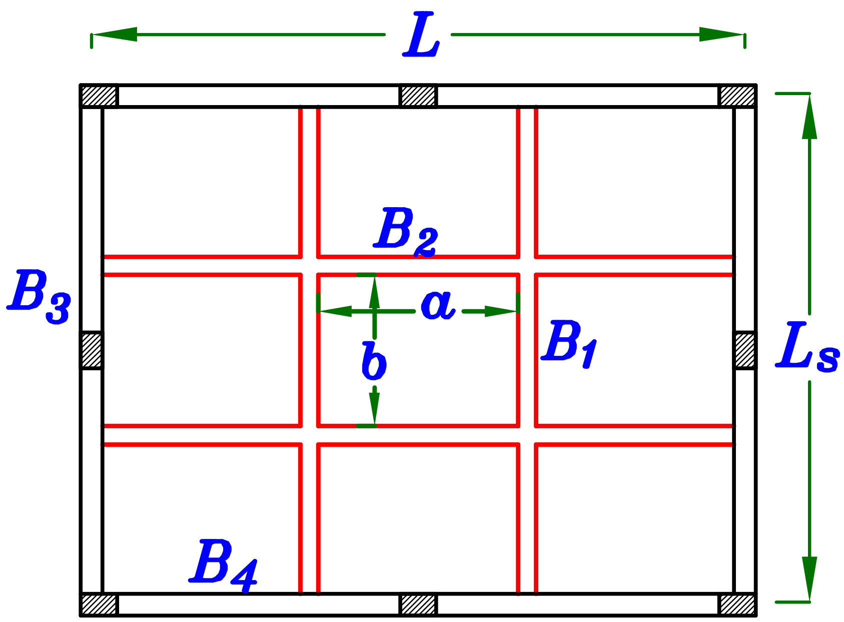
Simple Panelled Beams.



Steps of Design.

① **Design the slabs.** (Two way slab يفضل أن تكون)

(Solid or Hollow Blocks Slabs) (a * b)

a- Choose the Thickness of the Slab (t_s).

b- Get the Loads on the Slab (w_s).

c- Get the Load Factors α, β.

d- Take a strips in the slab (at the Load direction)

And then Get (B.M.) on the Slab & Design the slab.

e- Draw Details of RFT. of the slab in plan.

② **Design the Panelled Beams.** (B_1 , B_2)

a- Get Dimensions of the beams. (b,t).

b- Get the avarage Load on the Slab. (w_{av}).

c- Calculate α, β (For the hall area) By using Grashoff.

d- Get the Loads on the Panelled Beam & Calculate the B.M.

e- Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

F- Design the Panelled Beam.

g- Draw Details of RFT. For the Panelled Beams.

③ **Design the Edge Beams.**

(The Exterior Beam) (B_3 , B_4)

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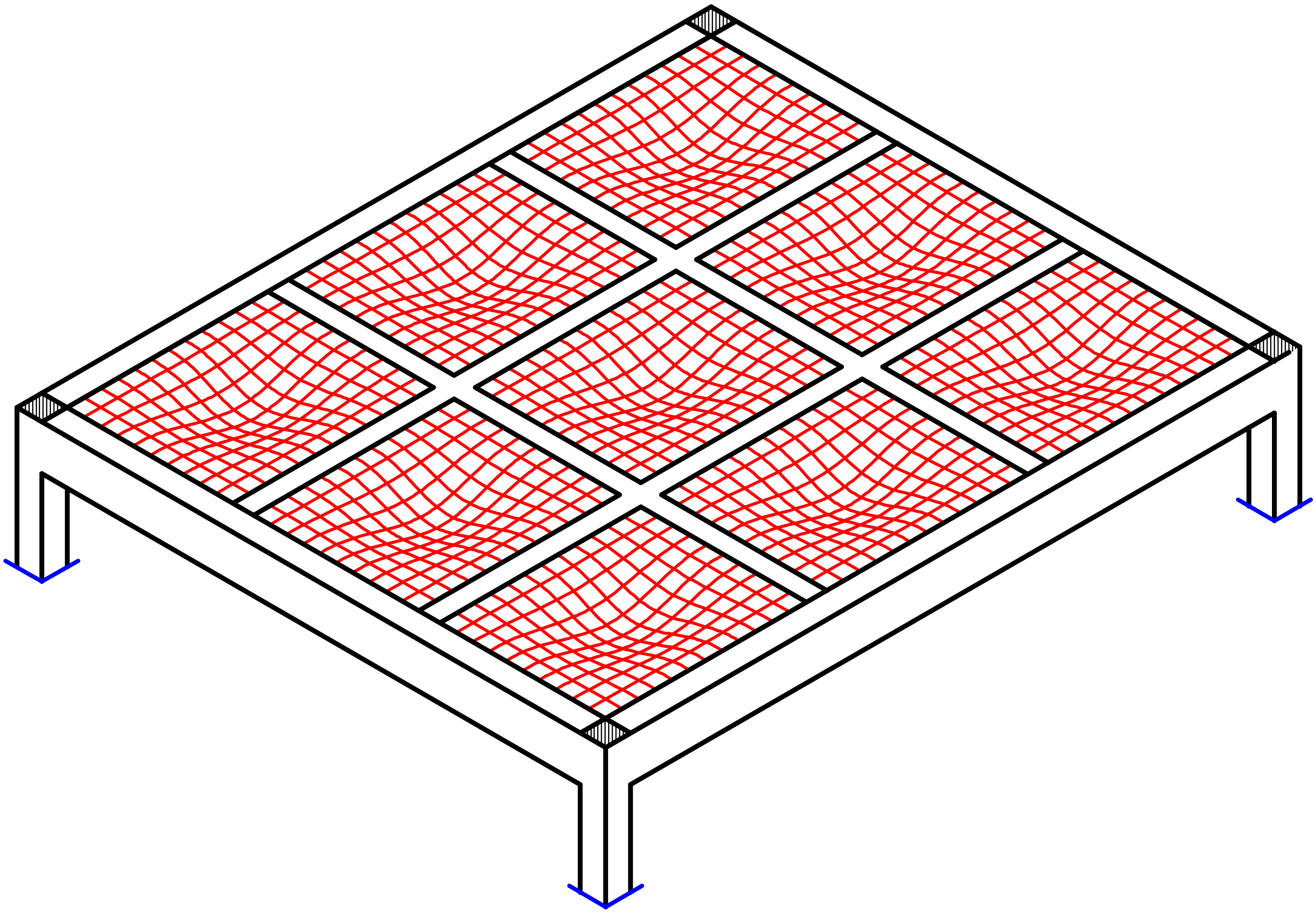
① Design the slabs.



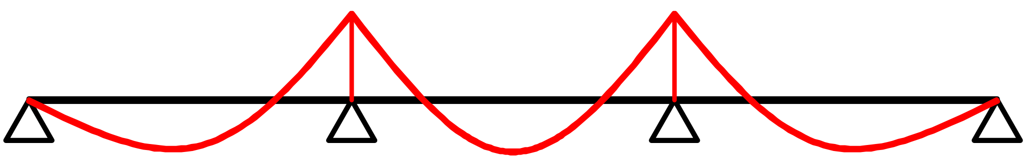
لان ال *Stiffness* للكمرات ال *Panelled* أكبر بكثير من البلاطه

اذا البلاطه تعتبر محموله على ال *Panelled Beams* و ال *Edge Beams*

اى اننا اعتبرنا الكمرات ال *Panelled* و ال *Edge Beams*

كأنها *Rigid Supports* للبلاطه .

لان شريحه البلاطه محموله على اكثر من *2 supports* اذا تعتبر شريحه *Continuous*



شريحه فى البلاطه

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يتم تصميم البلاطات سواء كانت *Solid or Hollow* بالخطوات العاديه لتصميم البلاطات .

IF all The Slabs are Two way Solid Slabs.

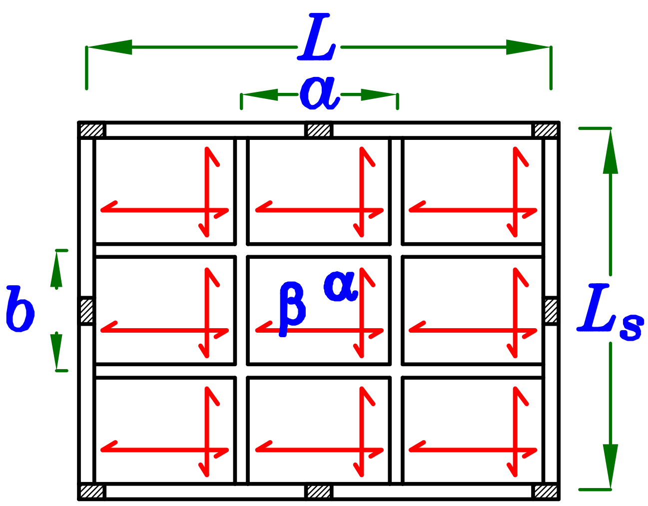
a- Choose the Thickness of the Slab. (t_s) .

Take $t_s = \dfrac{L_s}{35 \text{ or } 40 \text{ or } 45}$ where: $L_s = (b)$

b- Get the Loads on the Slab. (w_s).

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\ L.L.$$

c- Get the Load Factors α , β



الحالة العامة

Get $r = \dfrac{m\,L}{\grave{m}\,L_s} = \dfrac{m\,a}{\grave{m}\,b}$

① IF $L.L. \leqslant 5.0\ kN\backslash m^2$

Use C.P. (Code of Practice)

$$\boxed{\alpha = 0.5\,r - 0.15} \qquad \boxed{\beta = \frac{0.35}{r^2}}$$

② IF $L.L. > 5.0\ kN\backslash m^2$ Use Grashoff

$$\boxed{\alpha = \frac{r^4}{1+r^4}} \qquad \boxed{\beta = \frac{1}{1+r^4}}$$

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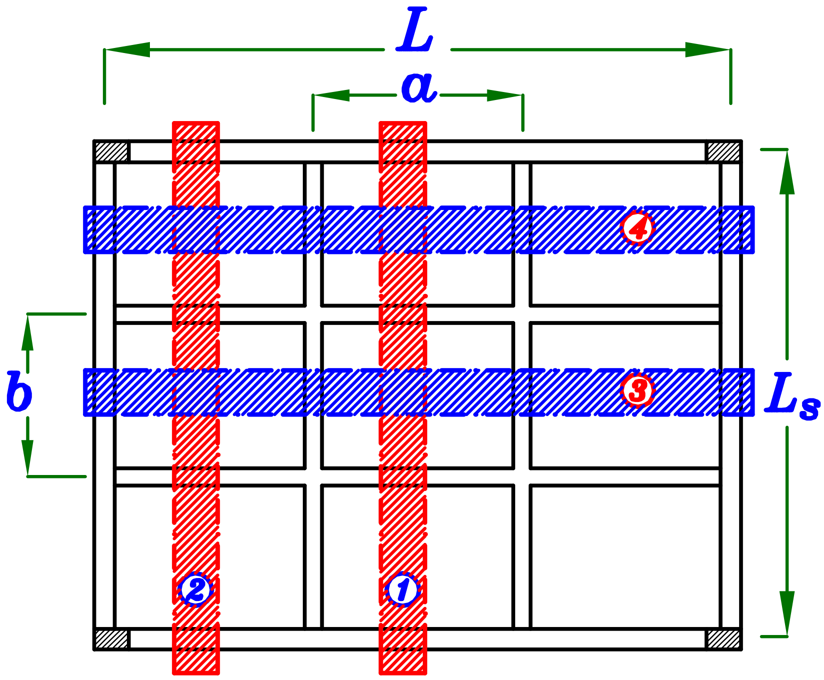
d- Take a strips in the slab (at the Load directions) And then Get the Bending Moments (B.M.) on the Slab. & Design the slab.

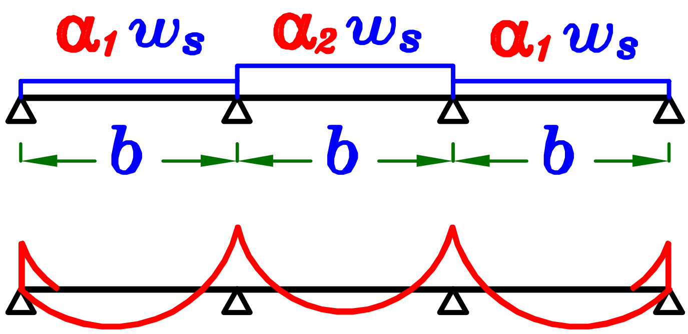
تؤخذ ال Strips

حسب شكل البلاطة

فى هذا الشكل يوجد 4 Strips



Strips at α direction.



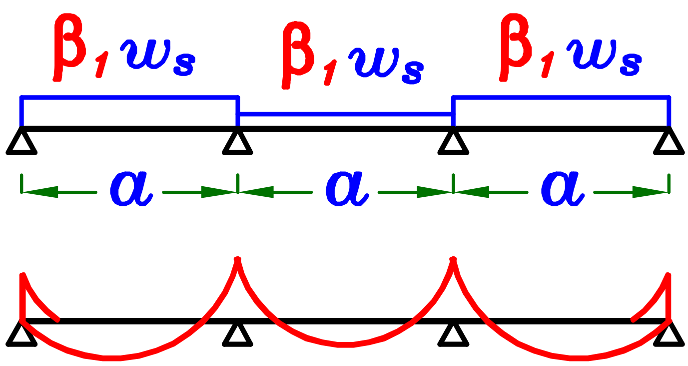
Take Cover For Short Dir.

$= 20$ mm (α Dir.)

$$d_{\alpha\ Dir.} = t_s - 20\ mm$$

Then get $A_{s\,\alpha}$

Strips at β direction.



Take Cover For Long Dir.

$= 30$ mm (β Dir.)

$$d_{\beta\ Dir.} = t_s - 30\ mm$$

Then get $A_{s\,\beta}$

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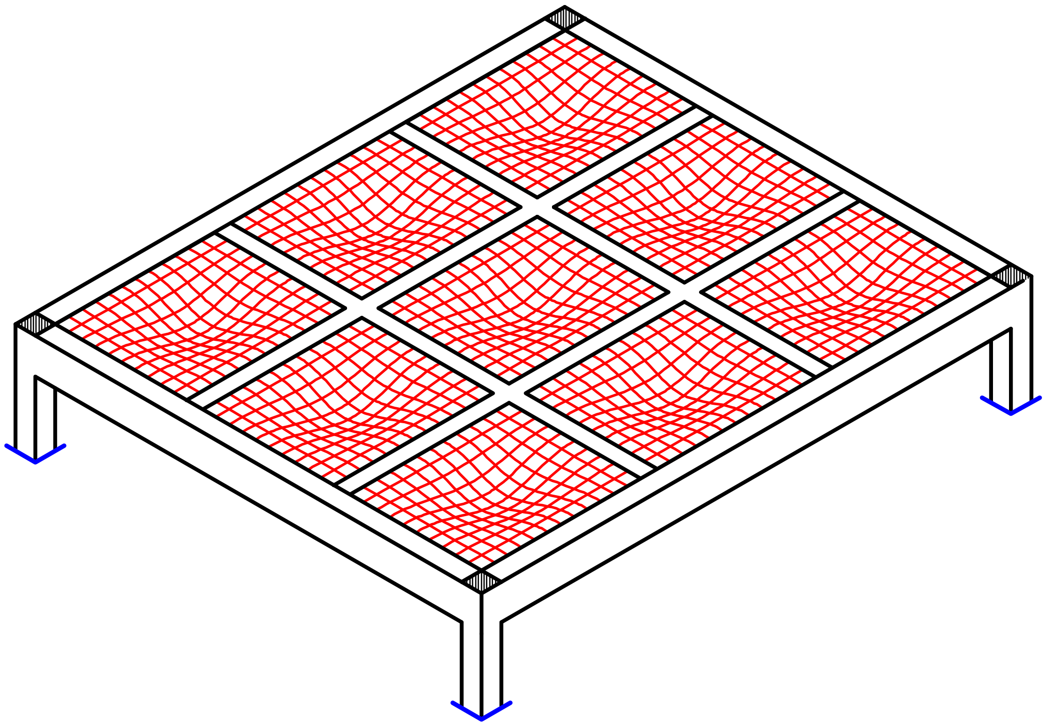
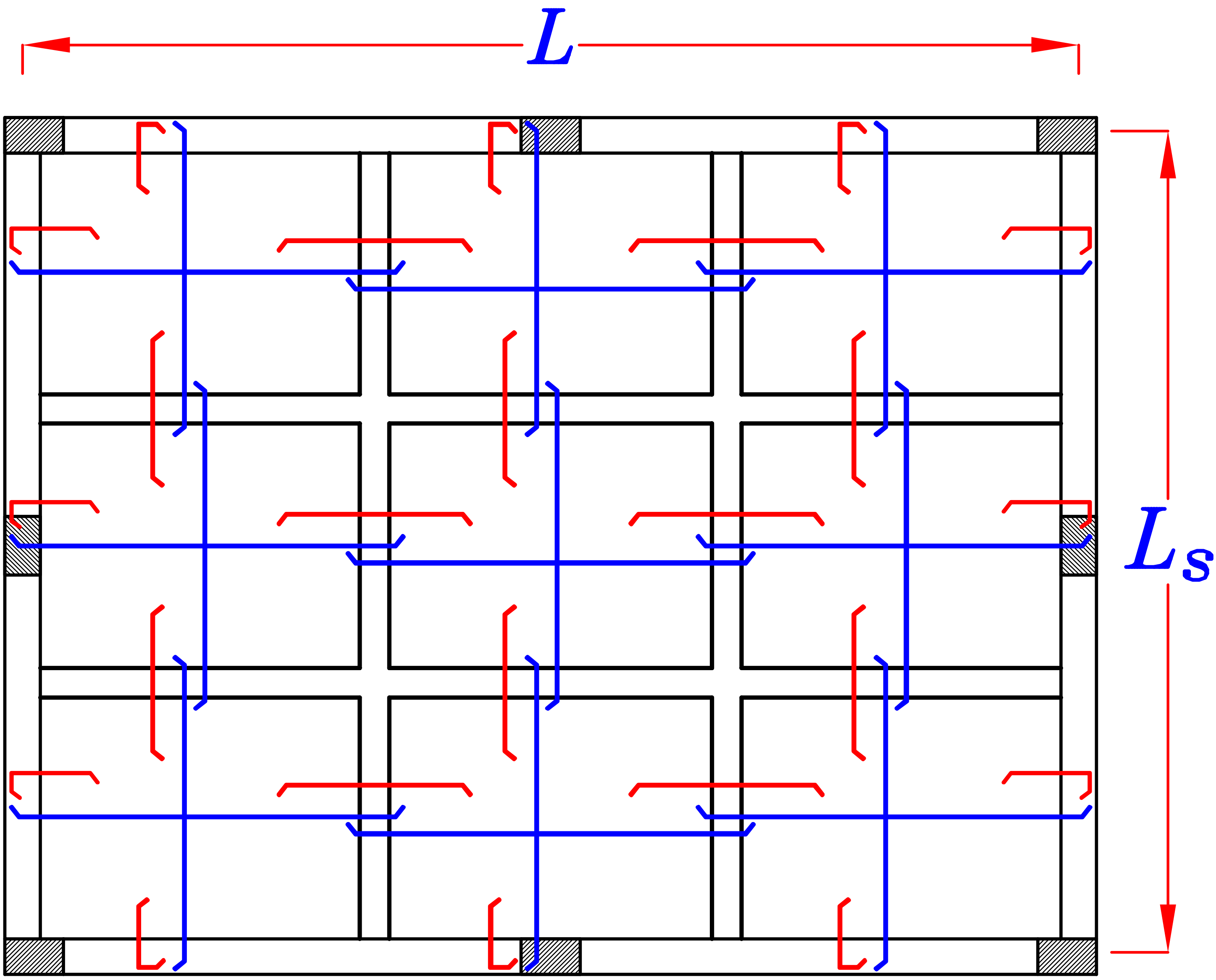
e – Draw Details of RFT. For the slab.





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② Steps of Design the Panelled Beams.

1 – Get Dimensions of the beams. (b, t).

2 – Get the average Load on the Slab. (w_{av}).

3 – Calculate α, β (For the hall area) By using Grashoff.

4 – Get Loads on the Panelled Beam & Calculate the B.M.

5 – Calculate the reduction Factor of the B.M. $\left(\frac{\sin \theta}{\sin 90}\right)$

6 – Design the Panelled Beam.

7 – Draw Details of RFT. For the Panelled Beams.

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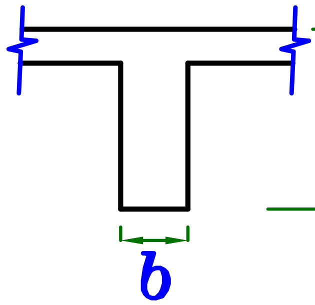
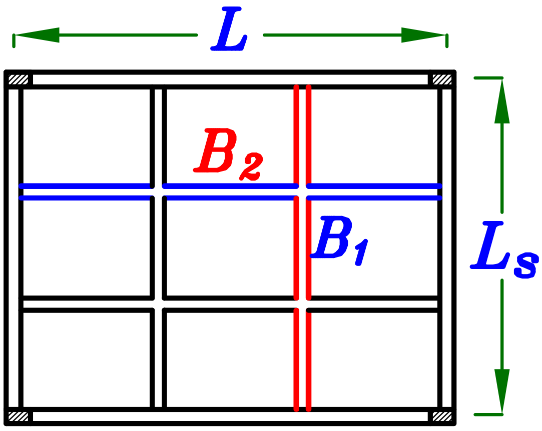
a - Get the Dimensions of the beam. (b, t)



- Take $b = 250\ mm$ or $= 300\ mm$



لان الكمرات B_1 , B_2 مكونه شبكه

اذاً عند وجود Load سيتوزع على مساحه اكبر

اذاً قيمه الـ Deflection للكمرات ستقل

اذاً قيمه الـ moment للكمرات ستقل

اذاً لن نحتاج قيمه t كبيره للكمرات .

- Take t For B_1 , B_2

$$\boxed{t = \frac{L_s}{16} = \checkmark\ mm}$$

تقرب لأقرب ٥٠ مم بالزياده

---

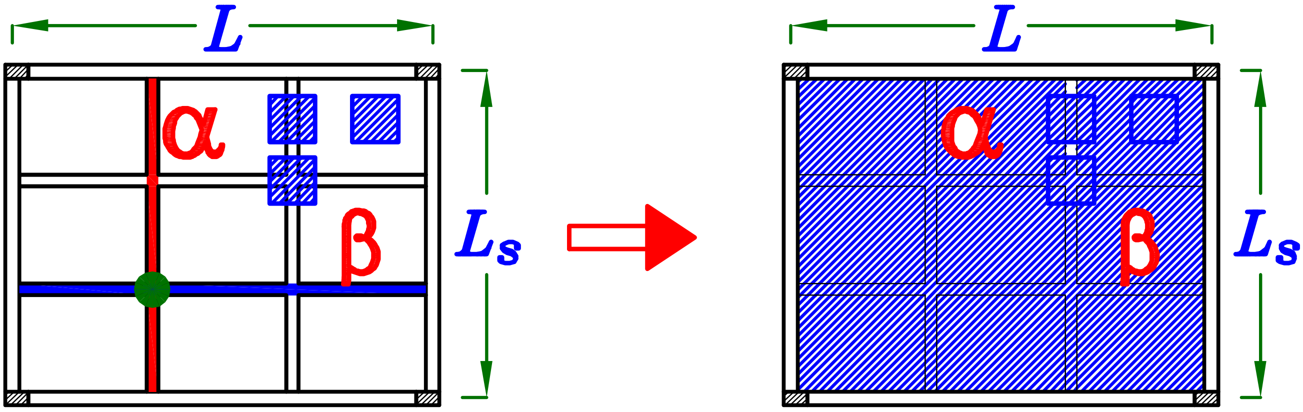
b - Get the avarage Load on the Slab. $(w_{av.})$

اذا وضع حمل عند تقاطع كمرتين معا سيتوزع على الكمرتين بنسبه α , β

و يتم حساب قيمتى α , β بطريقه Grashoff ولكن قوانين Grashoff مستنتجه

على أساس ان الحمل كله منتظم أى أن وزن المتر المربع متساوى على كل المساحه .

لذا نعمل على فرض ان الحمل كله منتظم بحساب قيمه $(w_{av.})$



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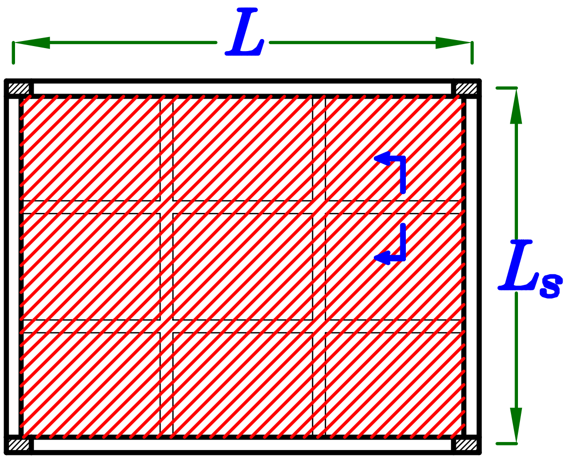
نعتبر أن السقف عباره عن بلاطة كبييره $(L * L_s)$
و لة وزن مكافئ $(w_{av.})$

$$w_{av.} = \frac{\sum Weight}{Area} = \checkmark \; kN\backslash m^2$$



$$w_{av.} = \frac{\text{Total Weight of slabs} + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

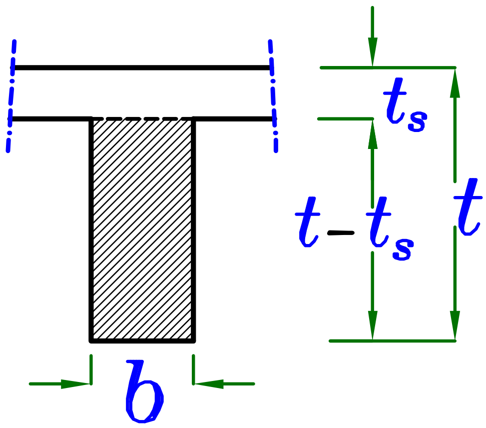
وزن المتر المربع من البلاطه

$$\text{Total Weight of slabs} = w_s * \text{Total area}$$
$$= w_s * (L * L_s)$$

Total Weight of Panelled Beams =

Volume of all Panelled Beams * γ_c

$b\,(t - t_s)$ [panelled مجموع أطوال الكمرات الـ] $* \gamma_c$

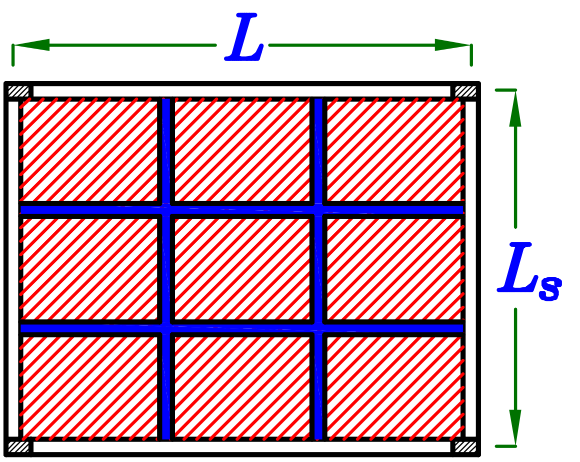


مجموع أطوال الكمرات الداخلية فى هذا المثال

$$\sum L = (2L + 2L_s)$$

Total Weight of Panelled Beams =

$$1.4 * b\,(t - t_s)\,(2L + 2L_s) * \gamma_c$$



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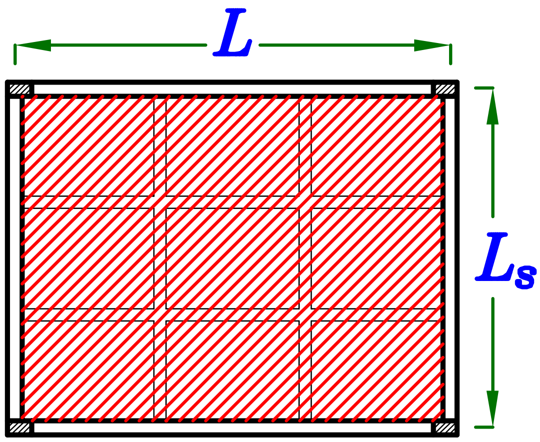
1- IF the slabs is Solid slabs only.

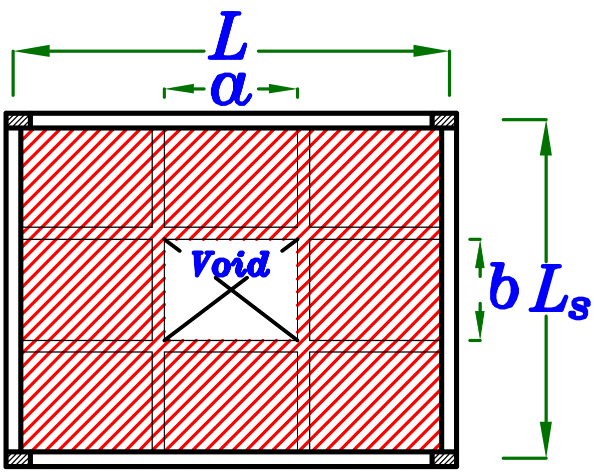


$$w_{av.} = \frac{\text{Total Weight of slabs} + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

$$w_{av.} = \frac{w_s * (L * L_s) + \text{Total Weight of Panelled Beams}}{L * L_s}$$

$$\boxed{w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s}}$$

2- IF the slabs is Solid slabs with Void.



$$\boxed{w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s - a * b}}$$

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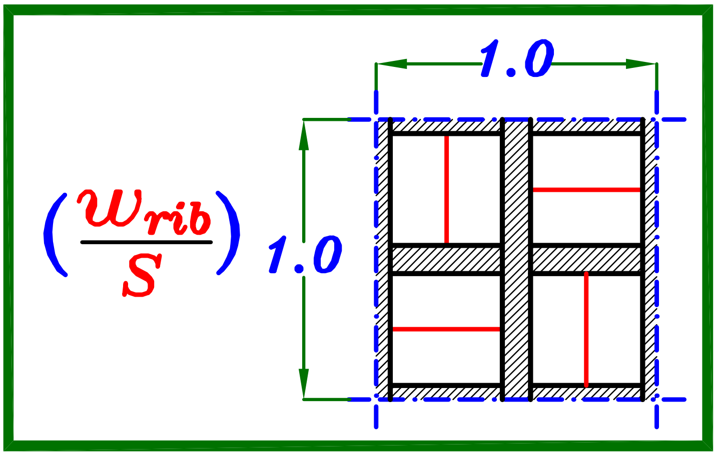
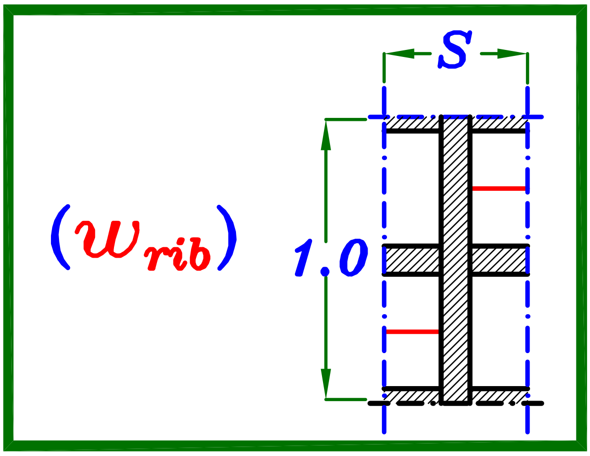
3 – IF the slabs is Solid & Hollow Blocks Slabs.

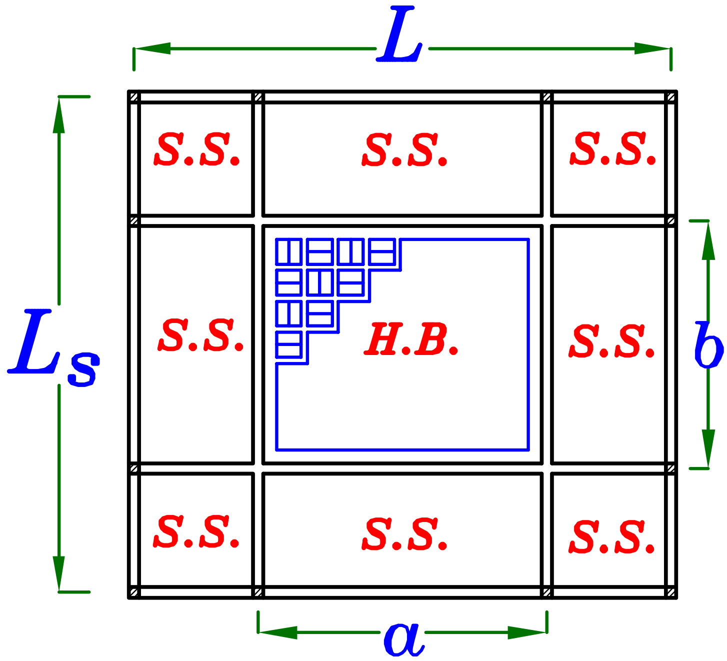
(W_{rib}) هو وزن مساحه $(1.0\ m * S)$ من البلاطه ال H.B.

∴ لحساب وزن $(1.0\ m * 1.0\ m)$ من البلاطه ال H.B. $= \left(\frac{W_{rib}}{S}\right)$





$$w_{av.} = \frac{\sum Weight}{Area} = \checkmark\ kN\backslash m^2$$



$$w_{av.} = \frac{\text{Total Weight of Solid slabs} + \text{Total Weight of H.B. slabs} + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

$$w_{av.} = \frac{w_s * area(s.s.) + \left(\frac{W_{rib}}{S}\right) * area(H.B.) + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

$$w_{av.} = \frac{w_s * (L * L_s - a * b) + \left(\frac{W_{rib}}{S}\right) * (a * b) + \left[1.4 * b(t - t_s)(2L + 2L_s) * \gamma_c\right]}{L * L_s}$$

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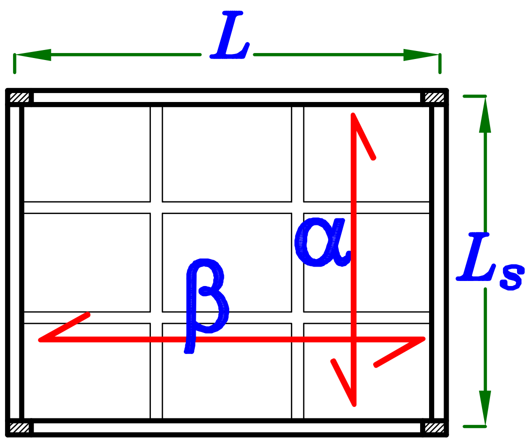
c- Calculate α , β By using Grashoff.

بعد تحويل الحمل كله الى حمل منتظم نستطيع الان ان نحسب قيم β , α عن طريق **Grashoff**

و لاننا نحسب قيم β , α للكمرات الـ **Panelled** و ليس للبلاطات

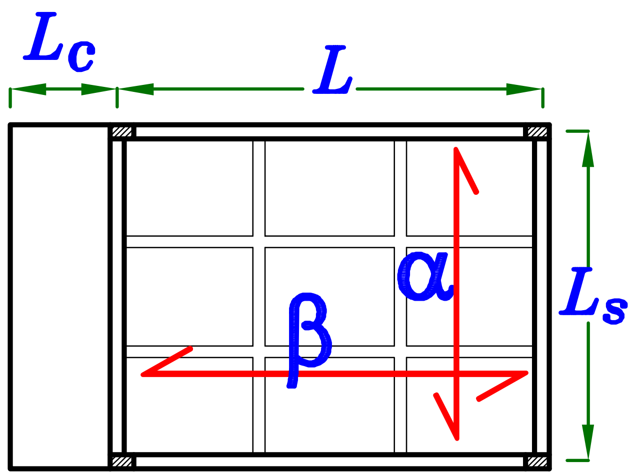
و لاننا ندرس الكمرات الـ **Simple Panelled Beams** فقط

اذا دائما ستكون قيمه $m , \grave{m}$ دائما تساوى 1.0



$$m = 1.0 \quad , \quad \grave{m} = 1.0$$

$$r = \frac{m \ L}{\grave{m} \ L_s} = \frac{(1.0) \ L}{(1.0) \ L_s}$$



$$m = 1.0 \quad , \quad \grave{m} = 1.0$$

$$r = \frac{m \ L}{\grave{m} \ L_s} = \frac{(1.0) \ L}{(1.0) \ L_s}$$

نستخدم **Grashoff** لحساب كلا من β , α

Grashoff

$$\boxed{\alpha = \frac{r^4}{1+r^4}} \qquad \boxed{\beta = \frac{1}{1+r^4}}$$

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d- Get the Loads on the Beams & Calculate The B.M.

لحساب الاحمال فوق كل كمره Panelled لن نستطيع عمل Load Distribution

لانه فى الكمرات ال Panelled لا يوجد كمرات تحمل الاخرى

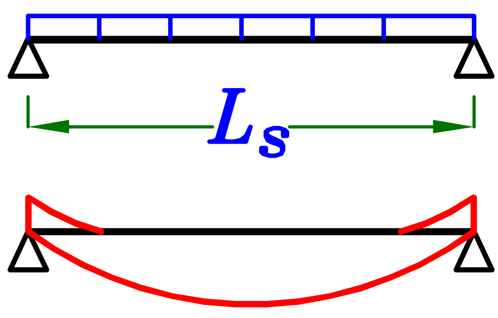
اى ان الحمل الكلى يتوزع على الكمرات فى الاتجاهين بنسب α , β

لذا نعتبر كل كمره تحمل نفس احمال الشريحه التى عرضها من C.L. البلاطه على يمين الكمره

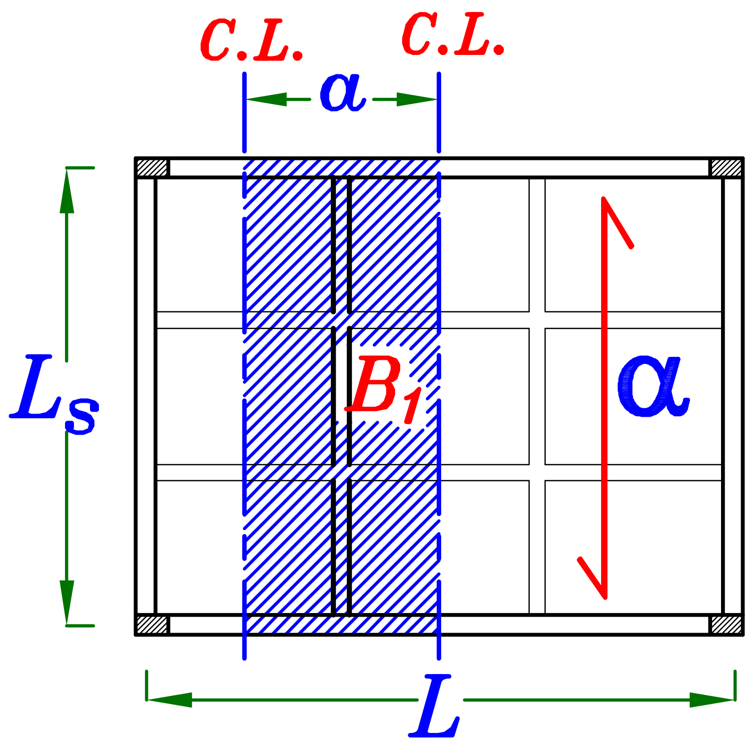
الى C.L. البلاطه على يسار الكمره .

For B_1

$$\boxed{w_1 = w_{av.} * \alpha * \alpha}$$



M * Reduction Factor



$$\boxed{M_1 = \frac{w_1 * L_s^2}{8} * \text{Reduction Factor}}$$

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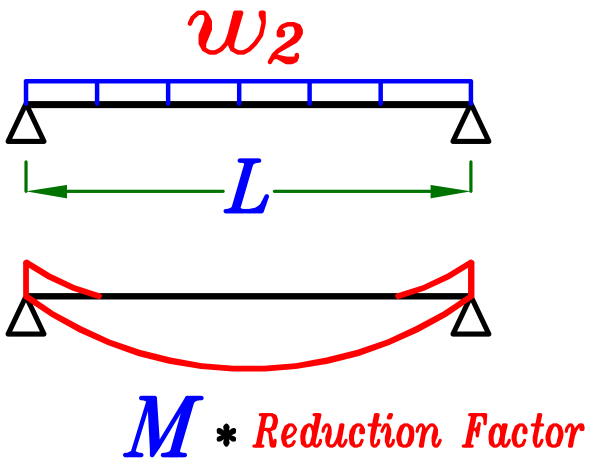
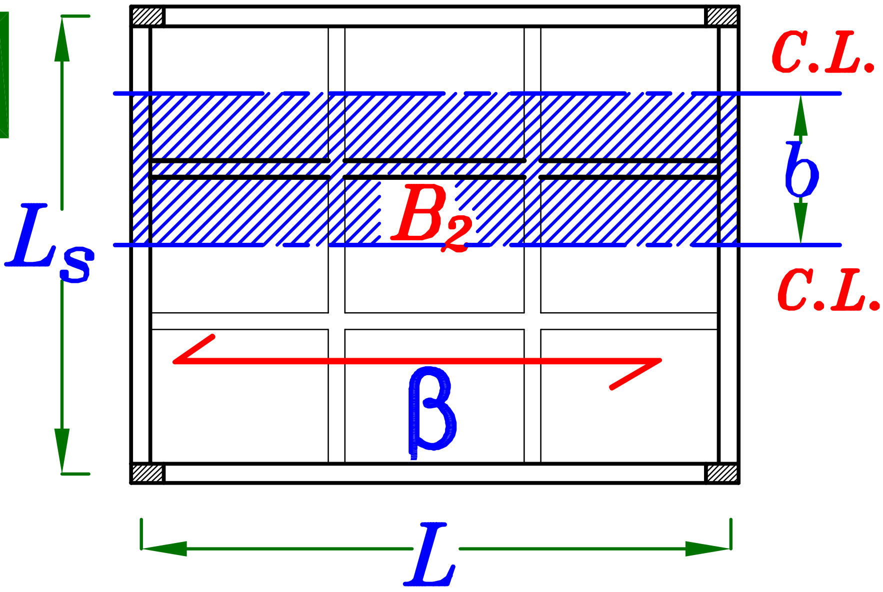
For B_2

$$w_2 = w_{av.} * b * \beta$$

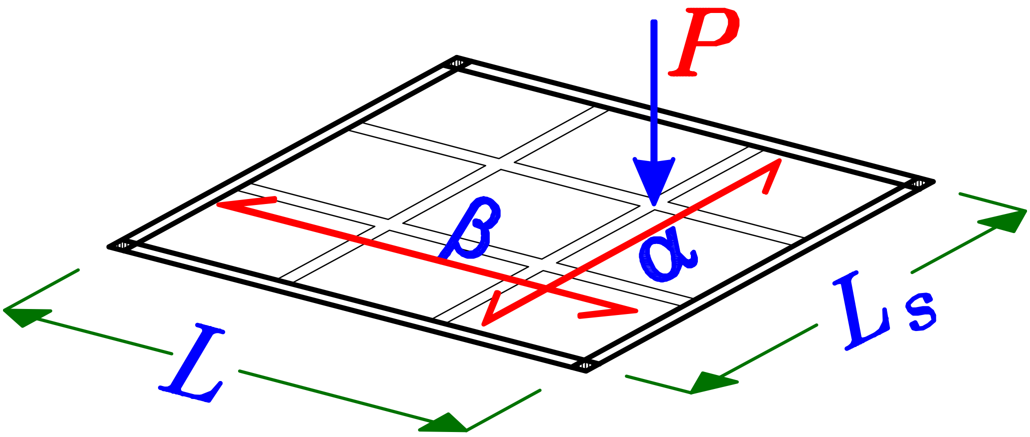


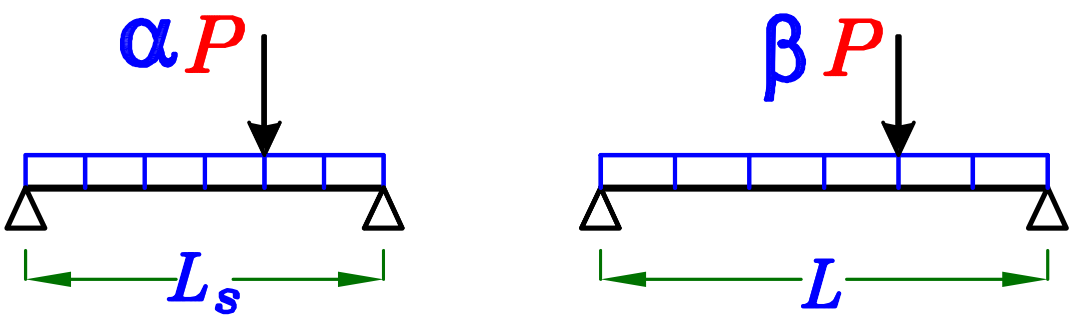


$$M_2 = \frac{w_2 * L^2}{8} * \text{Reduction Factor}$$

ملحوظه

عند وجود حمل مركز (P) واقع على تقاطع كمرتين يتوزع على الكمرتين بنفس نسب α , β





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e- Calculate the reduction Factor of the B.M.

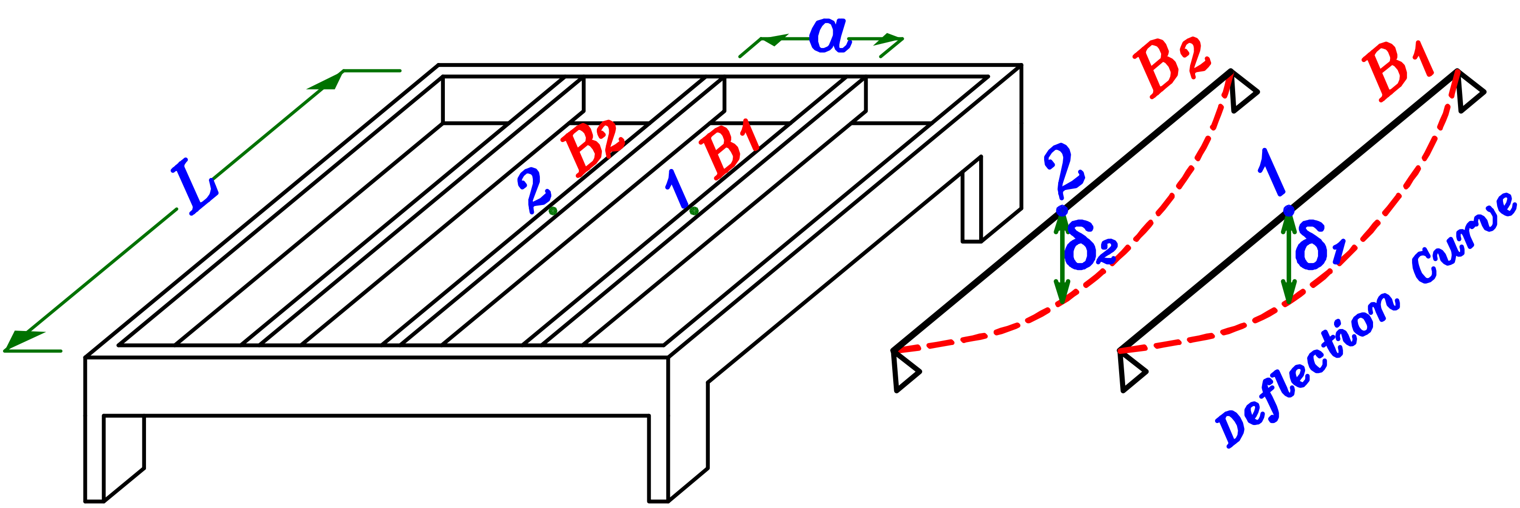
بعد حساب ال moment على الكمرات ال panelled

نحتاج لتقليل هذا ال moment لوجود الكمرات فى شبكه Grid Action

و لفهم تأثير ال Grid Action و تحديد قيمه ال Reduction Factor

نشرح فى المثال التالى :

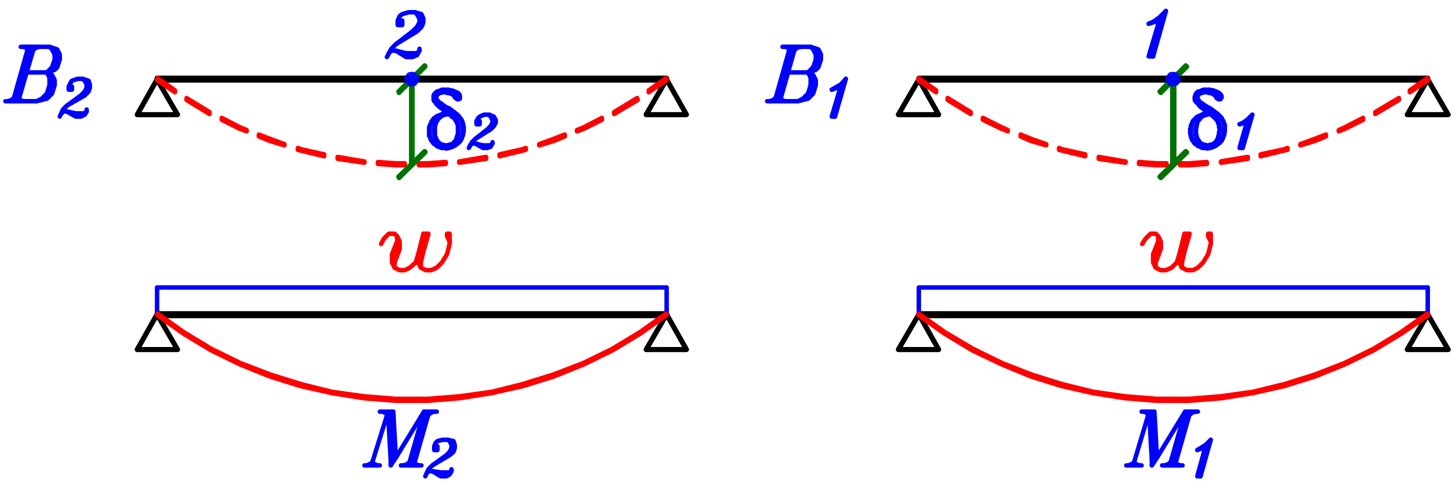
اذا كانت الكمرات B_1 , B_2 موضوعه على مسافات متساويه a



$$w = w_1 = w_2 = w_{av.} * a$$ ستحمل الكمرتين نفس الحمل

و بالتالى سيكون ال Deflection للكمرتين متساوى . $\because \boxed{\delta_1 = \delta_2}$

و بالتالى سيكون ال moment للكمرتين متساوى . $\therefore \boxed{M_1 = M_2}$



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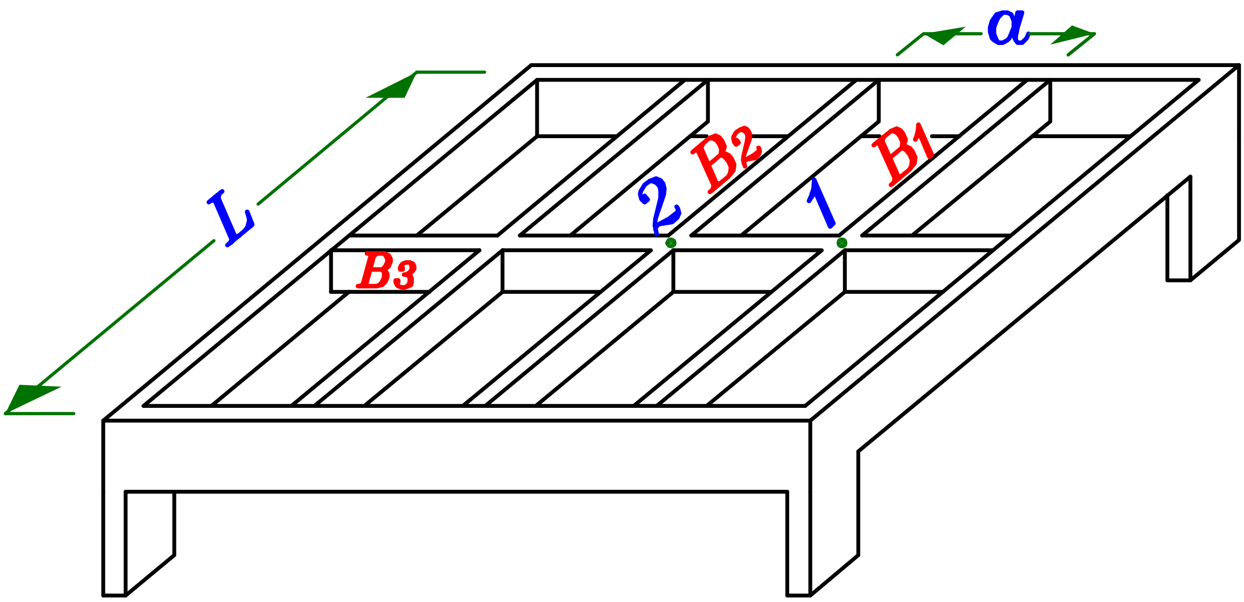
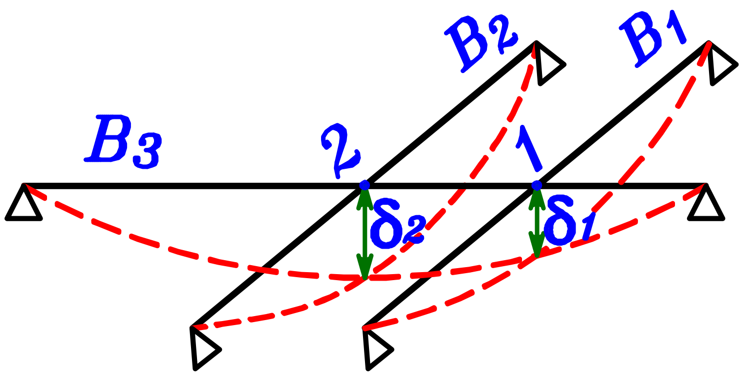
اذا وضعنا الكمره B_3 عموديه على الكمرتين B_1, B_2 فى المنتصف تماماً .

أى اننا ربطنا الكمرتين B_1, B_2 بكمره عموديه عليهم .





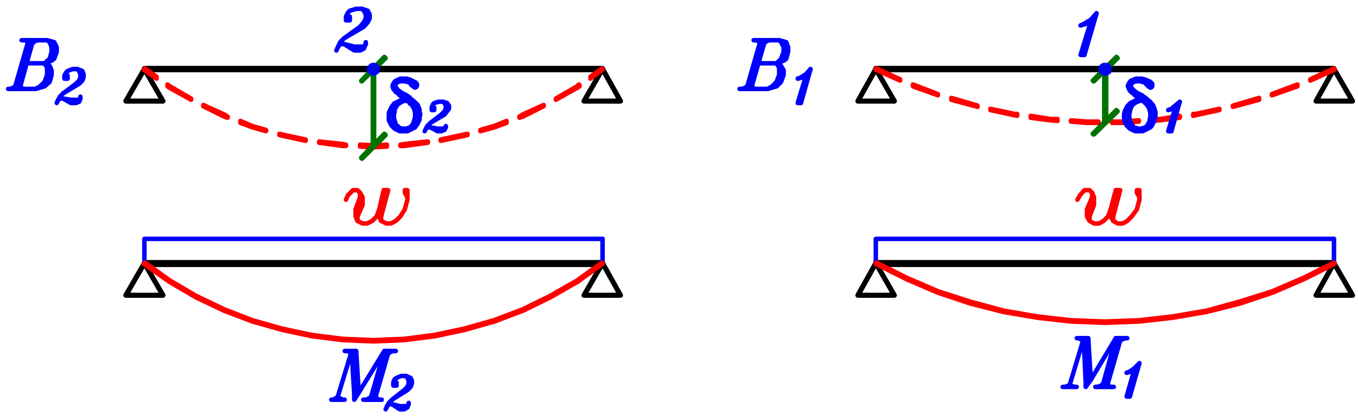
Deflection Curve

فى هذه الحاله سيكون ال Deflection للكمره B_1 أقل من ال Deflection للكمره B_2 التى فى المنتصف .

ستحمل الكمرتين نفس الحمل $w = w_1 = w_2 = w_{av.} * a$

و لكن Deflection الكمرتين غير متساوى . $\therefore \boxed{\delta_1 < \delta_2}$

و بالتالى سيكون ال moment للكمرتين غير متساوى . $\therefore \boxed{M_1 < M_2}$



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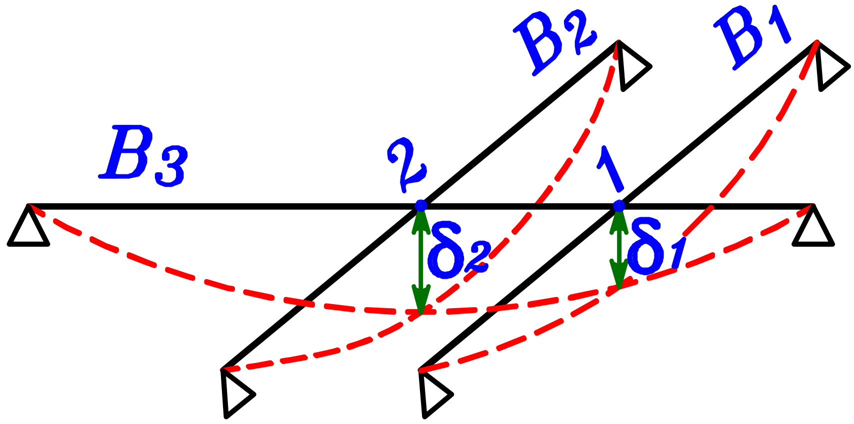
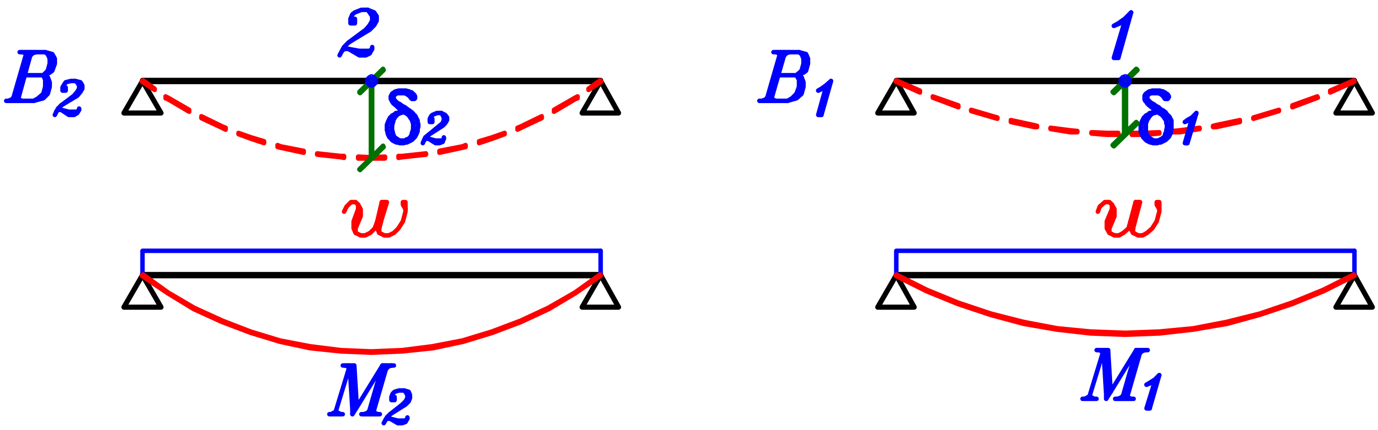


و لحساب قيمه ال **Reduction Factor** للكمره B_1 يساوى $\left(\frac{\delta_1}{\delta_2}\right)$

حيث δ_1 هى قيمه ال **Deflection** عند تقاطع الكمره المطلوبه B_1 مع الكمره العموديه عليها B_3

حيث δ_2 هى قيمه أكبر **Deflection** عند الكمره العموديه عليها B_3



$$M_2 = w * \frac{L^2}{8}$$ ------ لا يوجد لها **Reduction Factor** لانها فى المنتصف تماما

$$M_1 = w * \frac{L^2}{8} * \text{Reduction Factor}$$

Where $$\boxed{\text{Reduction Factor} = \left(\frac{\delta_1}{\delta_2}\right)}$$

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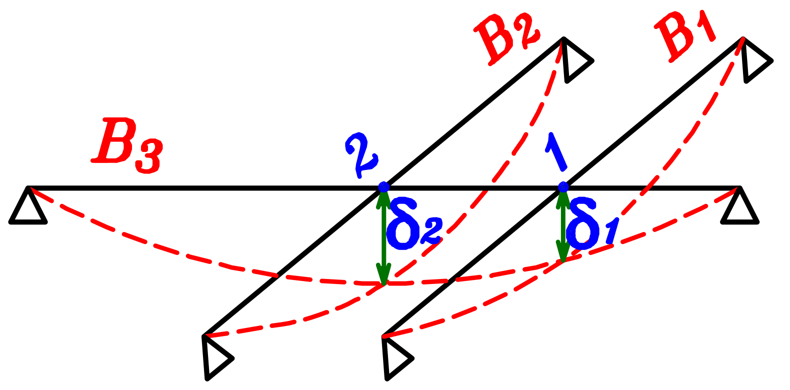
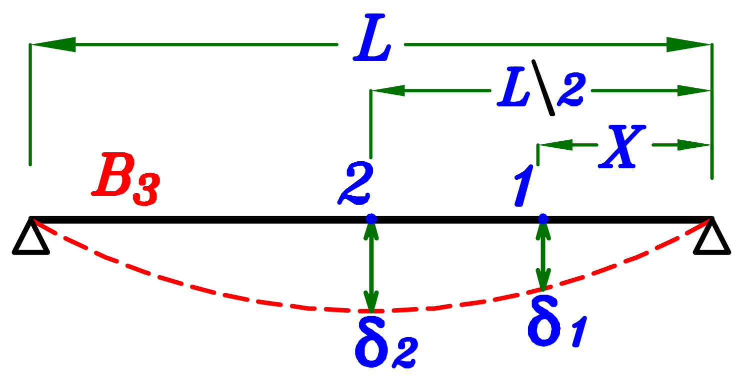
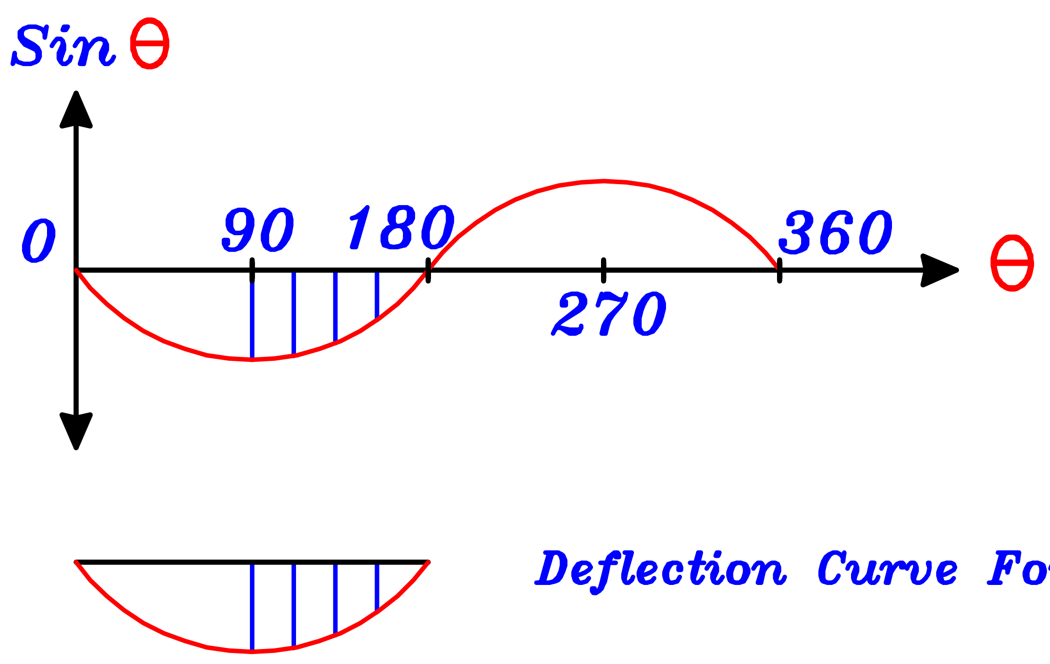
و لحساب قيمه الـ Reduction Factor للكمره B_1 يساوى $\left(\frac{\delta_1}{\delta_2}\right)$

نحتاج لحساب النسبه $\left(\frac{\delta_1}{\delta_2}\right)$ من شكل الـ Deflection للكمره B_3 (الكمره العموديه على الكمره المطلوبه)





وجد ان شكل الـ Sin Curve يشبه تماما شكل الـ Deflection Curve للكمرات الـ Simple



Sin Curve

Deflection Curve For Simple Beam

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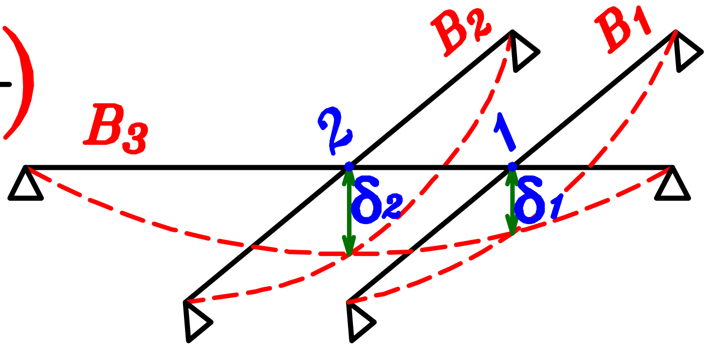
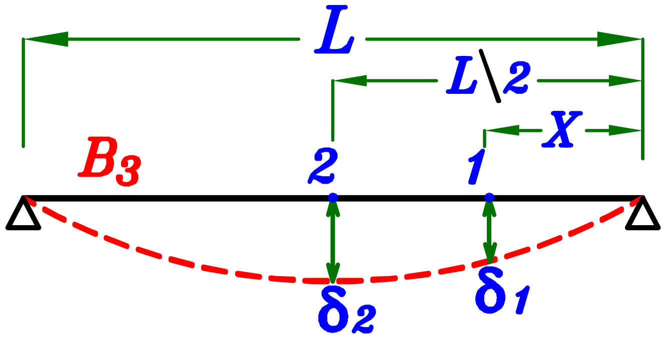
To Calculate

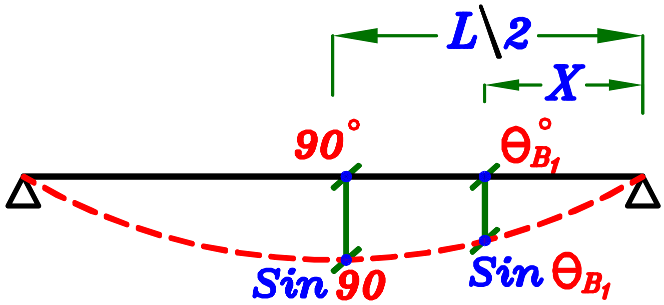
The Reduction Factor $= \left(\dfrac{\delta_1}{\delta_2}\right)$



Deflection Curve



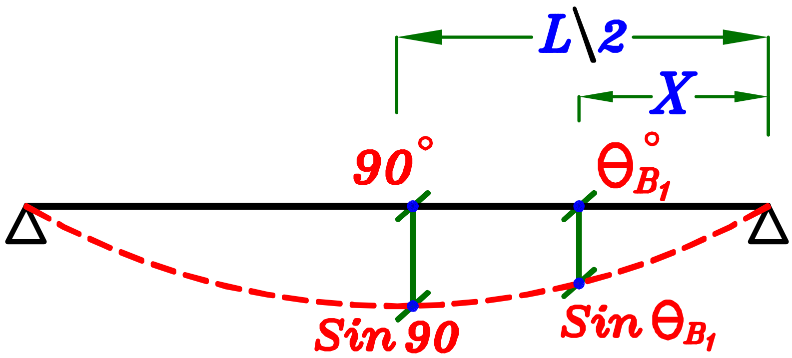
Sin. Curve



$$\therefore \boxed{\textbf{Reduction Factor} = \left(\frac{\delta_1}{\delta_2}\right) = \frac{\sin \theta_{B_1}}{\sin 90}}$$

To Calculate θ_{B_1}

$$\frac{\theta_{B_1}}{90} = \frac{X}{L \backslash 2}$$



Sin. Curve

$$\therefore \boxed{\theta_{B_1} = \frac{X}{(L \backslash 2)} * 90^{\circ}}$$

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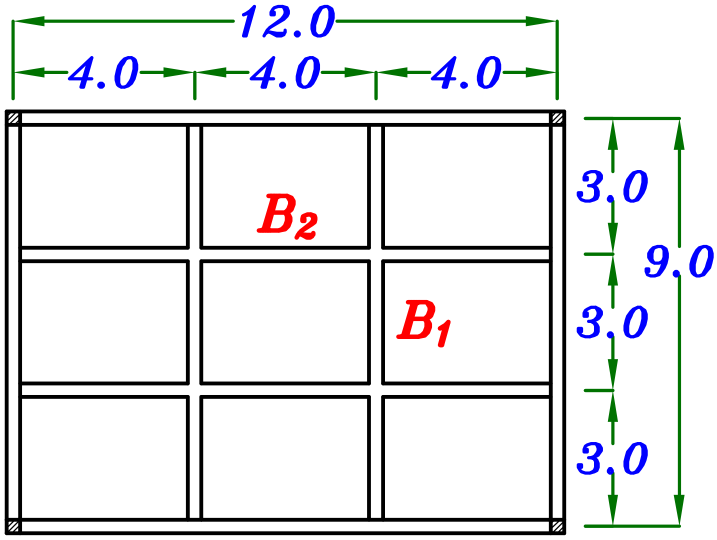
Examples.



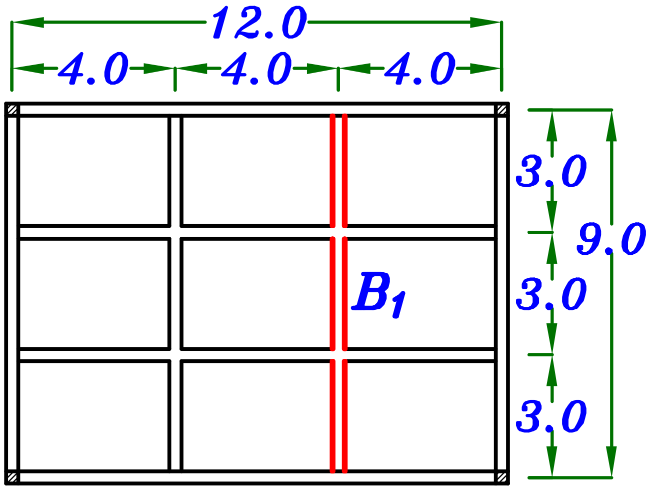
Calculate the **Reduction Factor** For Beams B_1, B_2

B_1

$X = 4.0\ m \quad , \quad \frac{L}{2} = \frac{12.0}{2} = 6.0\ m$

$\theta_{B_1} = \frac{4.0}{6.0} * 90^\circ = 60^\circ$

$M_1 = w_1 * \frac{9^2}{8} * \frac{\sin 60^\circ}{\sin 90^\circ}$



ملحوظه ممكن اخذ $X = 8.0\ m$ بدلا من $4.0\ m$

فتكون قيمه $\theta_{B_1} = \frac{8.0}{6.0} * 90^\circ = 120^\circ$

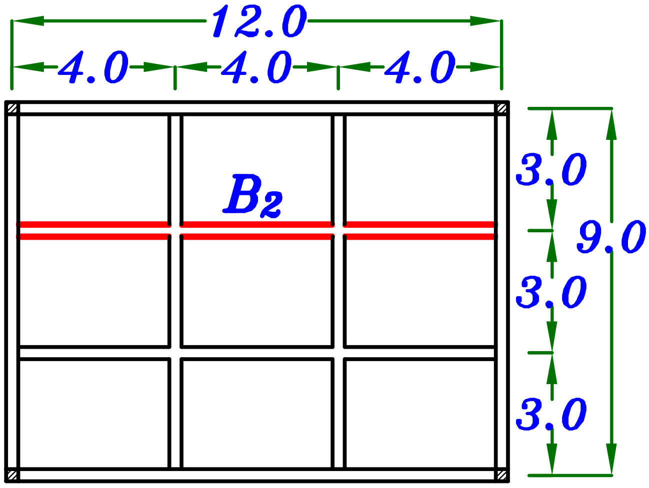
فتكون نفس النتيجه لان $\sin 60^\circ = \sin 120^\circ$

B_2

$X = 3.0\ m \quad , \quad \frac{L}{2} = \frac{9.0}{2} = 4.5\ m$

$\theta_{B_2} = \frac{3.0}{4.5} * 90^\circ = 60^\circ$

$M_2 = w_2 * \frac{12^2}{8} * \frac{\sin 60^\circ}{\sin 90^\circ}$



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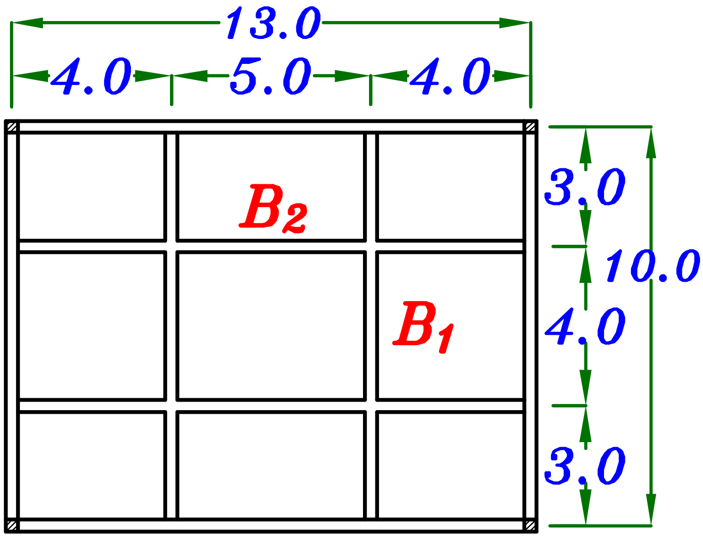
Examples.

Calculate the

Reduction Factor

For Beams B_1, B_2

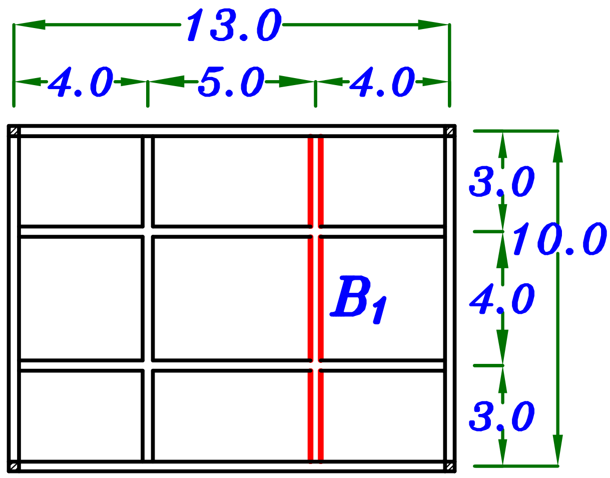


B_1

$X = 4.0\ m \quad , \quad \frac{L}{2} = \frac{13.0}{2} = 6.5\ m$

$\theta_{B_1} = \frac{4.0}{6.5} * 90^\circ = 55.38^\circ$

$M_1 = w_1 * \frac{10^2}{8} * \frac{\sin 55.38^\circ}{\sin 90^\circ}$

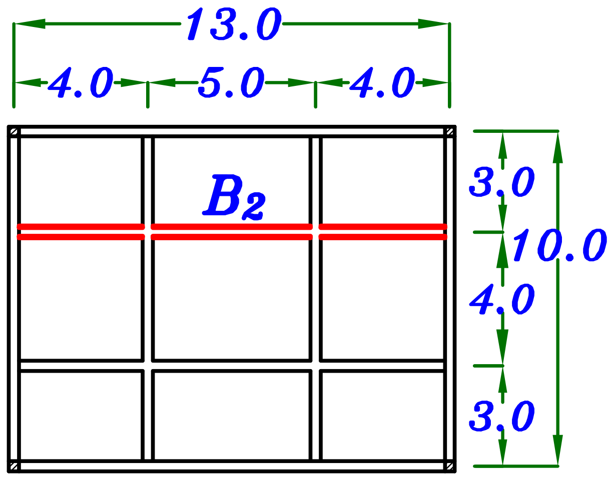


B_2

$X = 3.0\ m \quad , \quad \frac{L}{2} = \frac{10.0}{2} = 5.0\ m$

$\theta_{B_2} = \frac{3.0}{5.0} * 90^\circ = 54.0^\circ$

$M_2 = w_2 * \frac{13^2}{8} * \frac{\sin 54.0^\circ}{\sin 90^\circ}$



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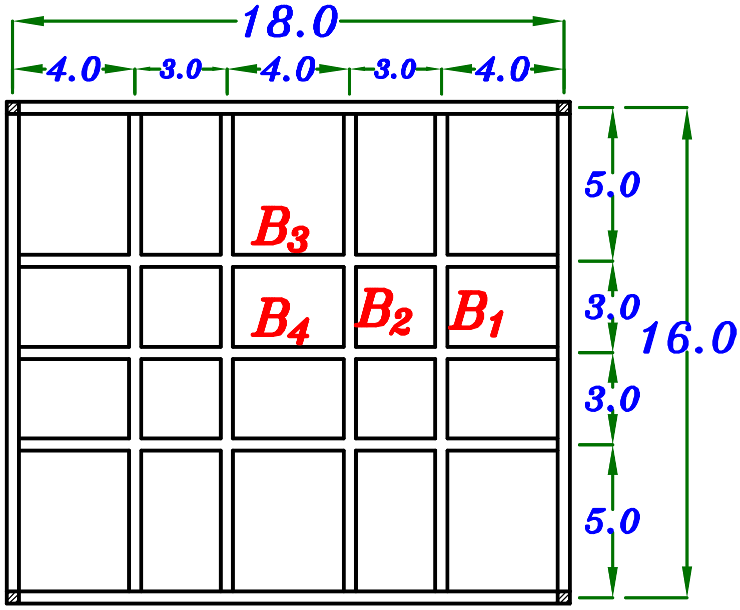
Examples.

Calculate the **Reduction Factor** For Beams B_1, B_2, B_3 & B_4

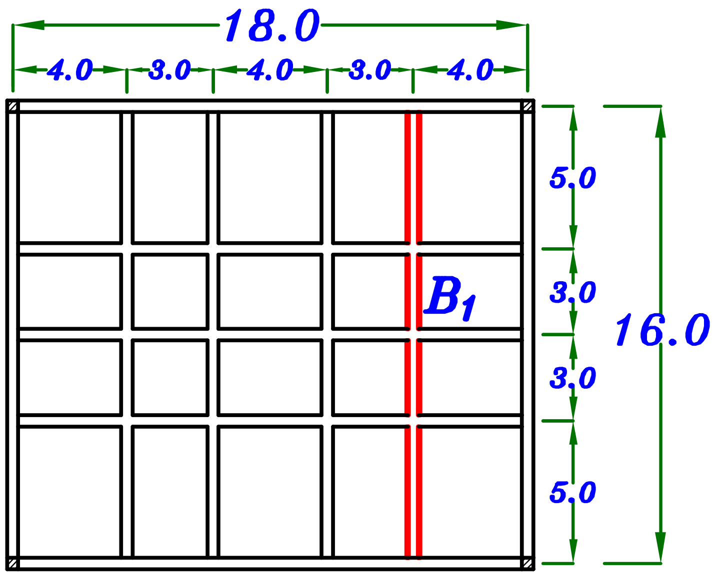


B_1

$$X = 4.0\ m \quad , \quad \frac{L}{2} = \frac{18.0}{2} = 9.0\ m$$

$$\theta_{B_1} = \frac{4.0}{9.0} * 90^\circ = 40^\circ$$

$$M_1 = w_1 * \frac{16^2}{8} * \frac{\sin 40^\circ}{\sin 90^\circ}$$

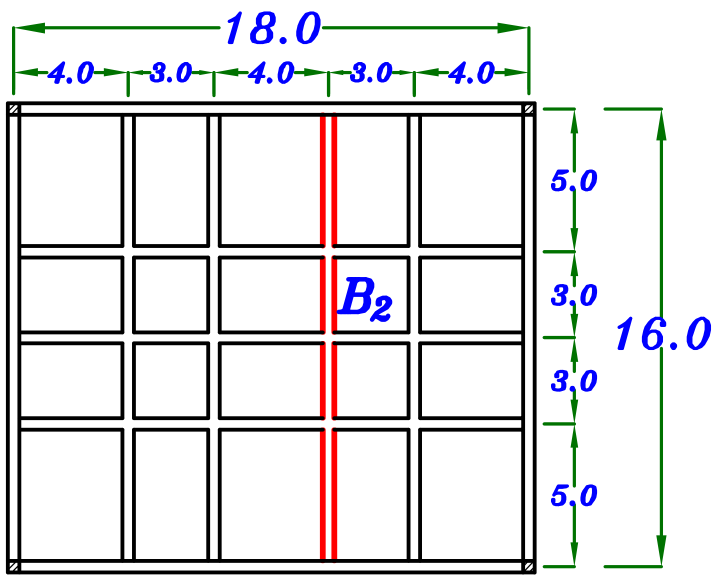


B_2

$$X = 7.0\ m \quad , \quad \frac{L}{2} = \frac{18.0}{2} = 9.0\ m$$

$$\theta_{B_2} = \frac{7.0}{9.0} * 90^\circ = 70^\circ$$

$$M_2 = w_2 * \frac{16^2}{8} * \frac{\sin 70^\circ}{\sin 90^\circ}$$



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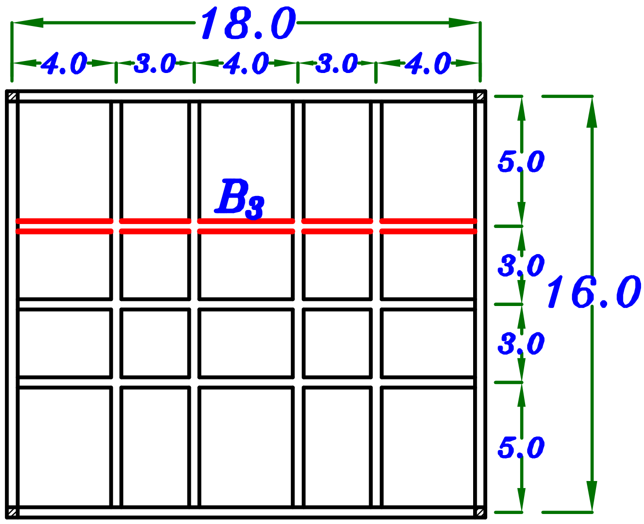
B_3

$$X = 5.0\ m\ ,\ \frac{L}{2} = \frac{16.0}{2} = 8.0\ m$$

$$\theta_{B_3} = \frac{5.0}{8.0} * 90^\circ = 56.25^\circ$$

$$M_3 = w_3 * \frac{18^2}{8} * \frac{\sin 56.25^\circ}{\sin 90^\circ}$$

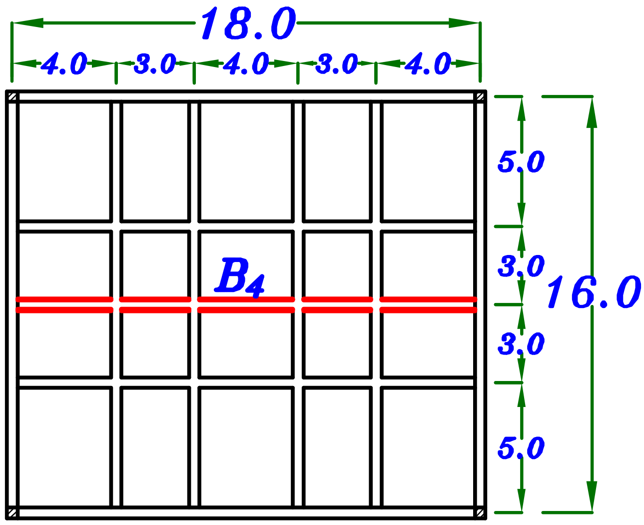


B_4

$$X = 8.0\ m\ ,\ \frac{L}{2} = \frac{16.0}{2} = 8.0\ m$$

$$\theta_{B_4} = \frac{8.0}{8.0} * 90^\circ = 90^\circ$$

$$M_4 = w_4 * \frac{18^2}{8} * \frac{\sin 90^\circ}{\sin 90^\circ}$$



ملحوظه

اذا كانت الكمره فى منتصف البحر تماما ستكون $\theta = 90^\circ$ أى أن $\frac{\sin\theta}{\sin 90} = 1.0$

أى لا يوجد *Reduction Factor* فتكون قيمه الـ *moment* كبيره

أى أنه لم يتم الاستفاده من الـ *Grid Action* فى تقليل الـ *moment*

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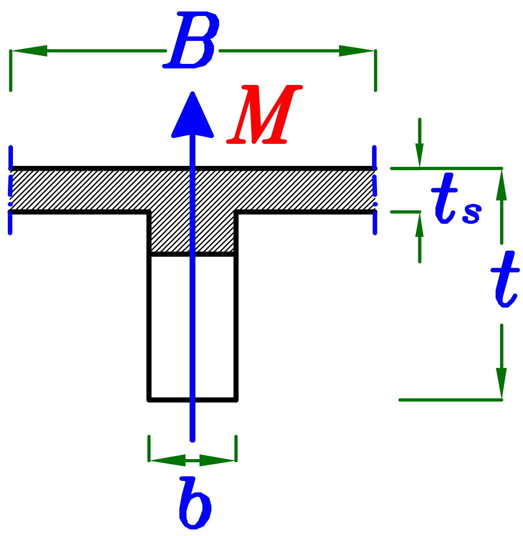
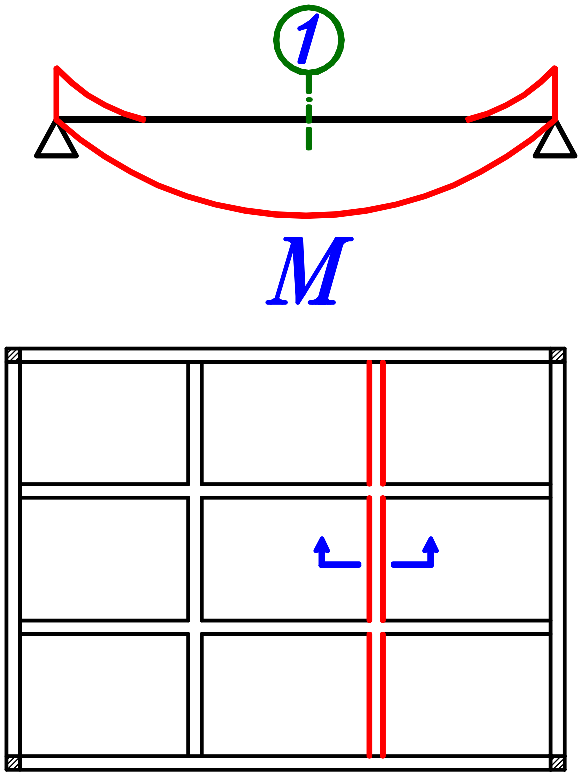
F – Design the Beam.

Take the cover = 50 mm α Direction

Take the cover = 70 mm β Direction





Design the Sec. as T–Sec.

$$B = \begin{cases} C.L. \rightarrow C.L. (\alpha \text{ or } \beta) \\ 16\, t_s + b \\ K \frac{L}{5} + b \quad (K = 1.0) \end{cases}$$

$$d = t - \text{Cover} = \checkmark$$

$$d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu} * B}} \longrightarrow C_1 = \checkmark \longrightarrow J = \checkmark$$

$$A_s = \frac{M_{U.L.}}{J\, F_y\, d} = \checkmark \ mm^2$$

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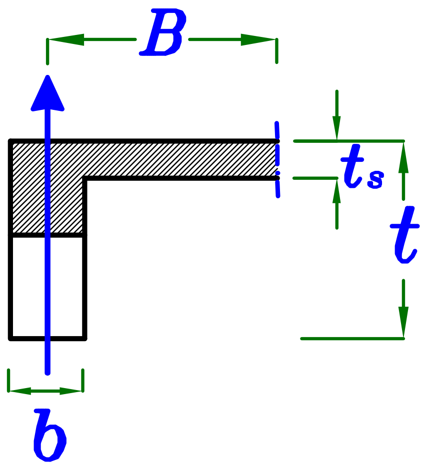
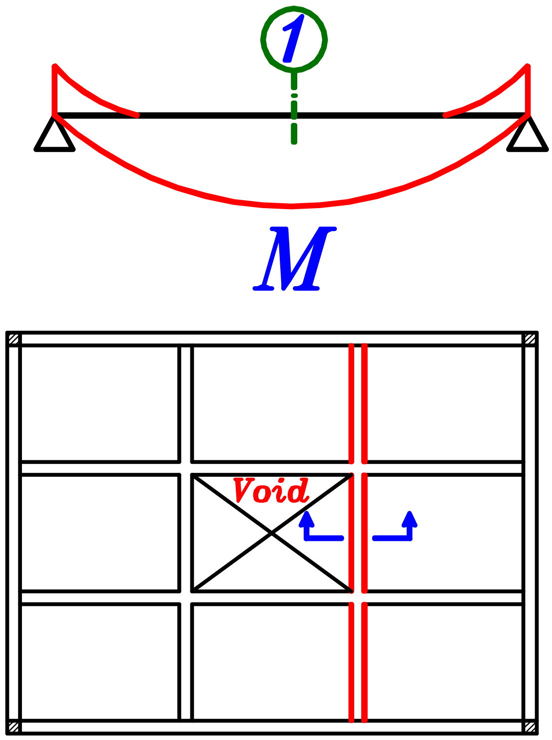
IF There is a Void.

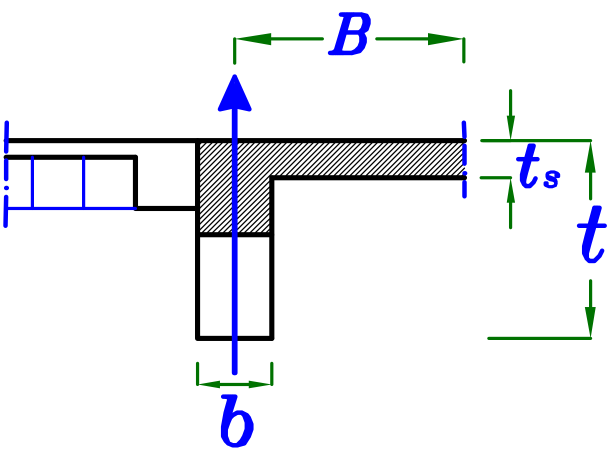


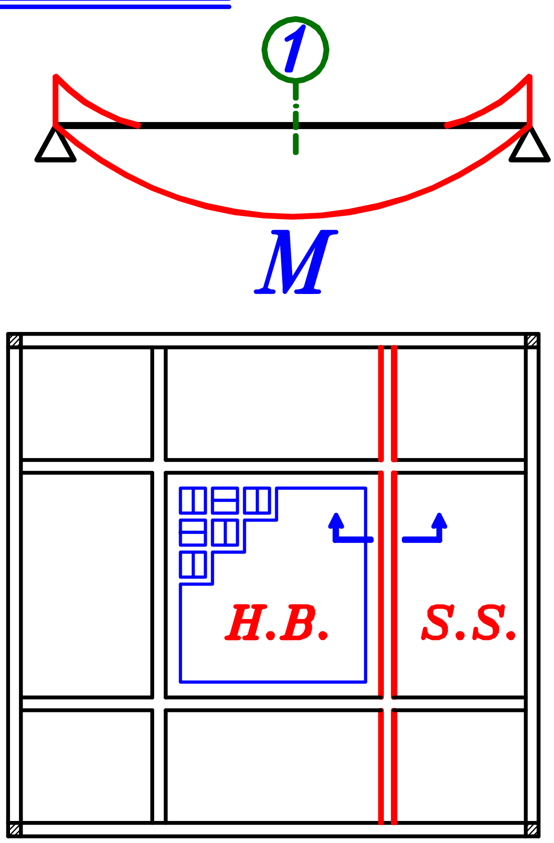


Design the Sec. as L-Sec.

$$B = \begin{cases} C.L._{beam} \rightarrow C.L._{Slab} \left(\frac{a}{2} \text{ or } \frac{b}{2}\right) \\ 6\, t_s + b \\ K \frac{L}{10} + b \quad (K = 1.0) \end{cases}$$

IF There is a Solid & Hollow Blocks.





Design the Sec. as L-Sec.

$$B = \begin{cases} C.L._{beam} \rightarrow C.L._{Slab} \left(\frac{a}{2} \text{ or } \frac{b}{2}\right) \\ 6\, t_s + b \\ K \frac{L}{10} + b \quad (K = 1.0) \end{cases}$$

يفضل تصميم القطاع على انه L-Sec. من جهه ال Solid Slab فقط.

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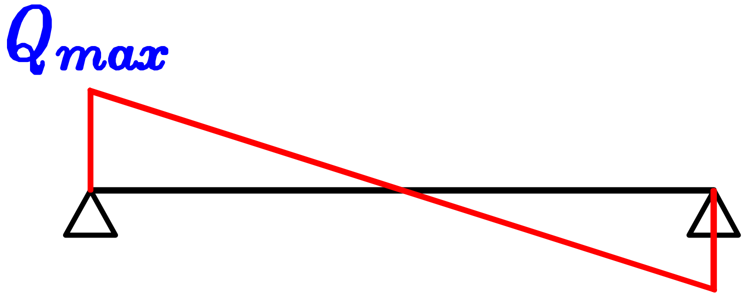
Check Shear.



* **Allowable Shear Stresses.**

$$q_{cu} = (0.24)\sqrt{\frac{F_{cu}}{\gamma_c}} \quad N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{F_{cu}}{\gamma_c}} \quad N\backslash mm^2$$

* **Actual Shear Stress.** $$\boxed{q_s = \frac{Q_{max}}{b\ d}} \quad (N\backslash mm^2)$$

1- IF $q_s < q_{cu}$ $\longrightarrow$ Use min. Shear RFT. $5\phi 8\backslash m'$

2- IF $q_s > q_{u_{max}}$ $\longrightarrow$ Increase Dimensions (b or d)

3- IF $q_{cu} < q_s < q_{u_{max}}$ $\longrightarrow$ $$\boxed{q_s - \frac{q_{cu}}{2} = \frac{n\, A_s (F_y\backslash \gamma_s)}{b\ S}}$$

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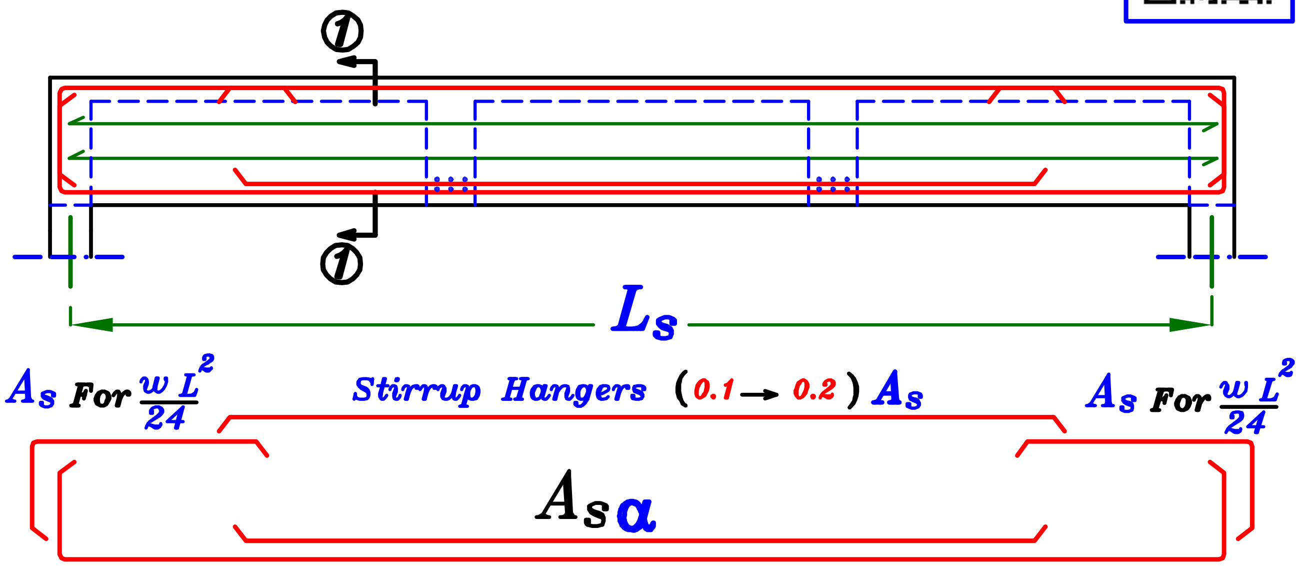
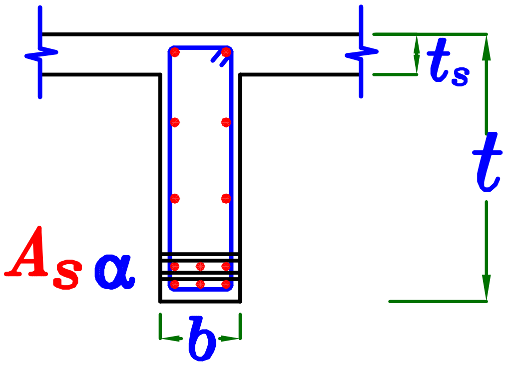
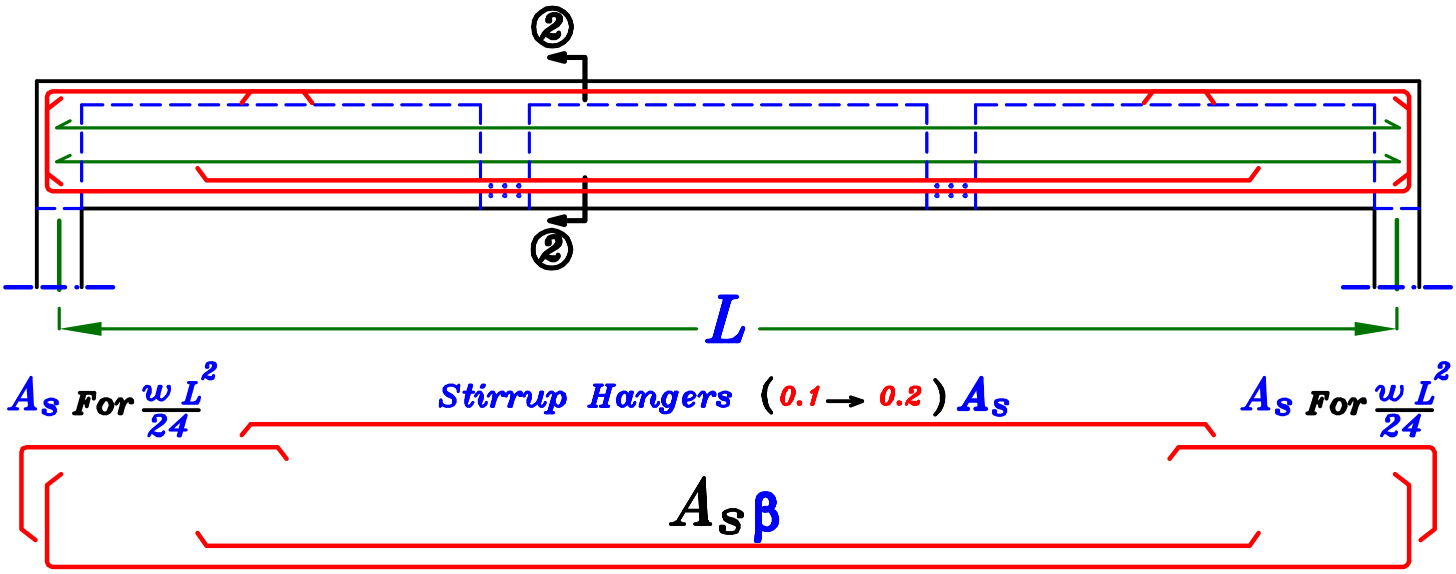
g- Draw Details of RFT. For the Beams.

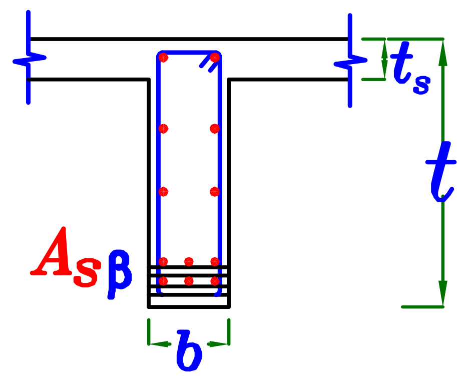


Sec. (1–1)





Sec. (2–2)



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③ Design of the Edge Beam. (Exterior Beam)

الكمرات الخارجيه محموله على أعمده أى انها محموله على **Rigid Support**

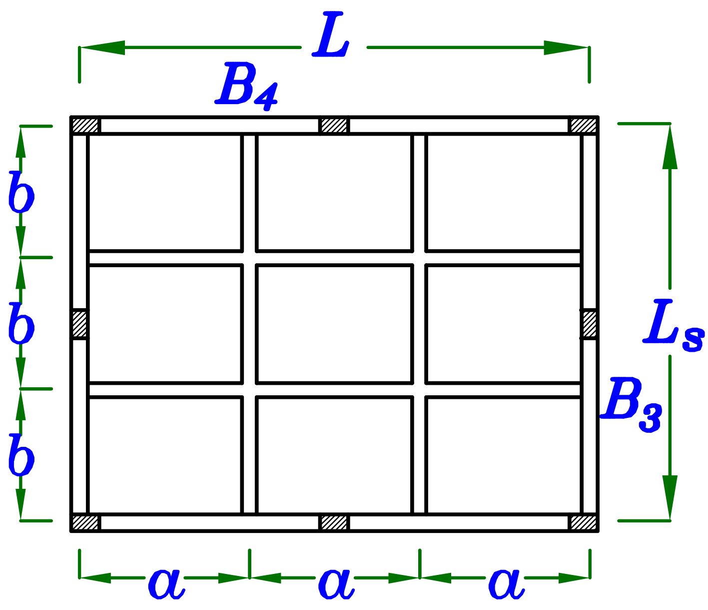
و ليست موجوده فى شبكه لذا لا يوجد بها **Reduction Factor**

و ممكن حساب الاحمال عليها عن طريق **Load Distribution**

و يتم تصميم الكمرات الخارجيه على الا **Shear & moment**

و لحساب الاحمال على الكمرات الخارجيه توجد حالتان :

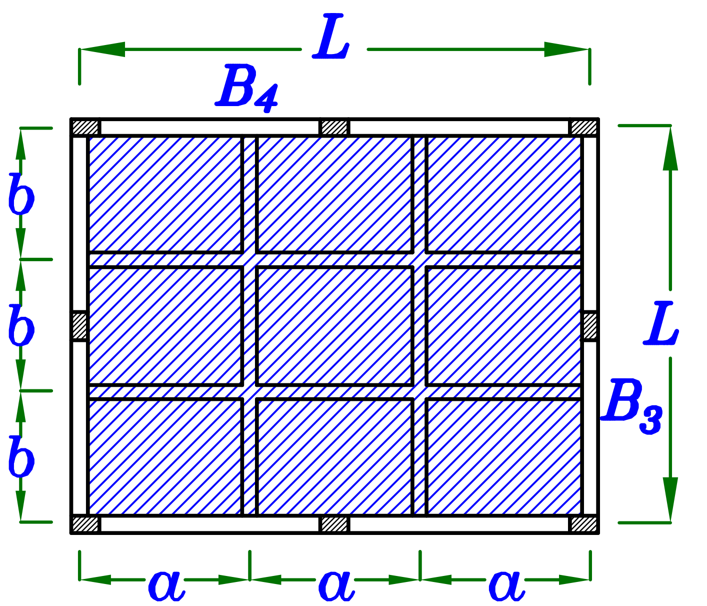
١- اذا كانت كل الكمرات الا *Panelled* محموله على الكمرات الخارجيه فقط.



ممكن استخدام حل **approximate**

و ذلك بأستخدام w_{av} لحساب الوزن

كأنها بلاطه كبيره أبعادها $(L * L_s)$



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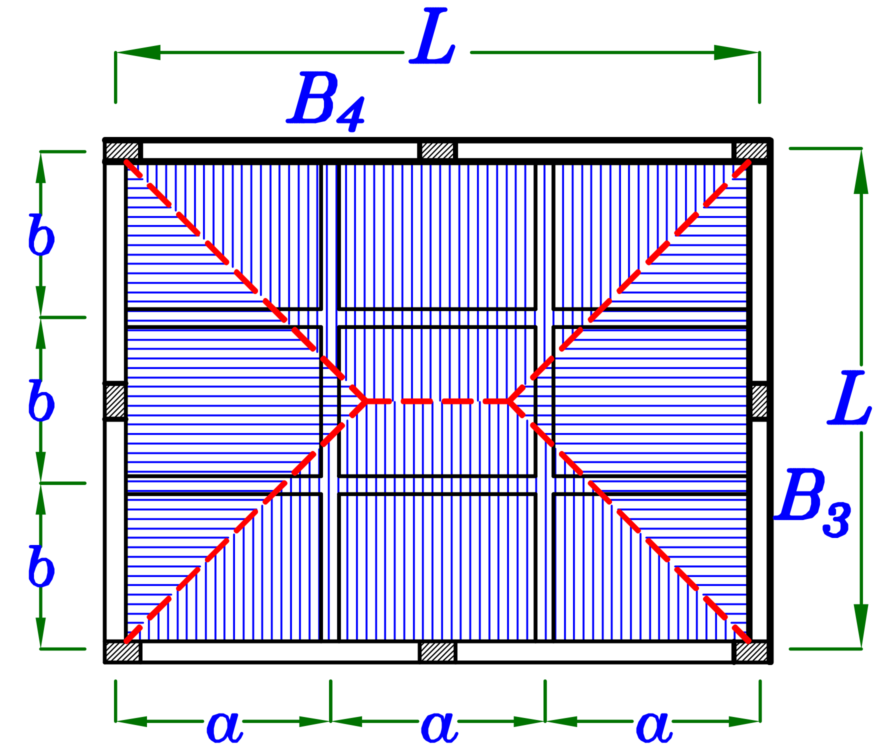
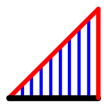
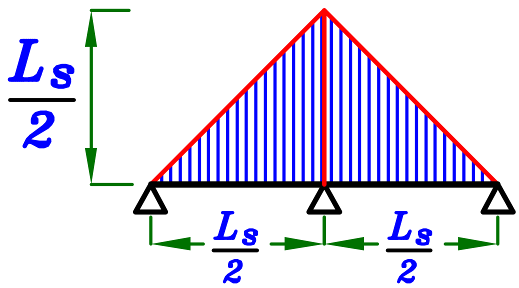
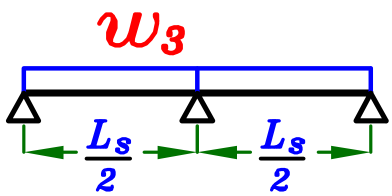


$\underline{B_3}$



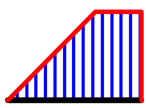


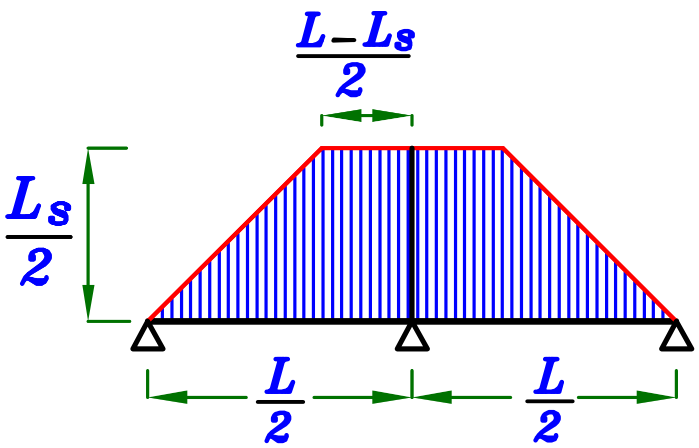
$$w_3 = (o.w.)_{beam} + \frac{\sum area}{span}\,(w_{av.})$$



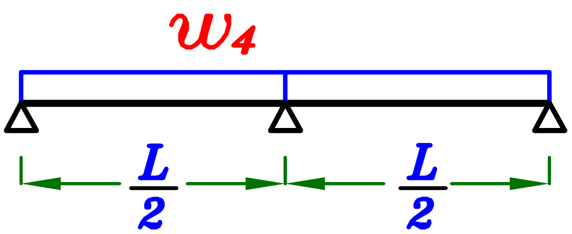
$$\frac{\sum area}{span} = \frac{\left(0.5 * \frac{L_s}{2} * \frac{L_s}{2}\right)}{L_s}$$

$\underline{B_4}$





$$w_4 = (o.w.)_{beam} + \frac{\sum area}{span}\,(w_{av.})$$



$$\frac{\sum area}{span} = \frac{\left[\dfrac{\left(\frac{L}{2} + \frac{L-L_s}{2}\right)}{2} * \frac{L_s}{2}\right]}{\left(\frac{L}{2}\right)}$$

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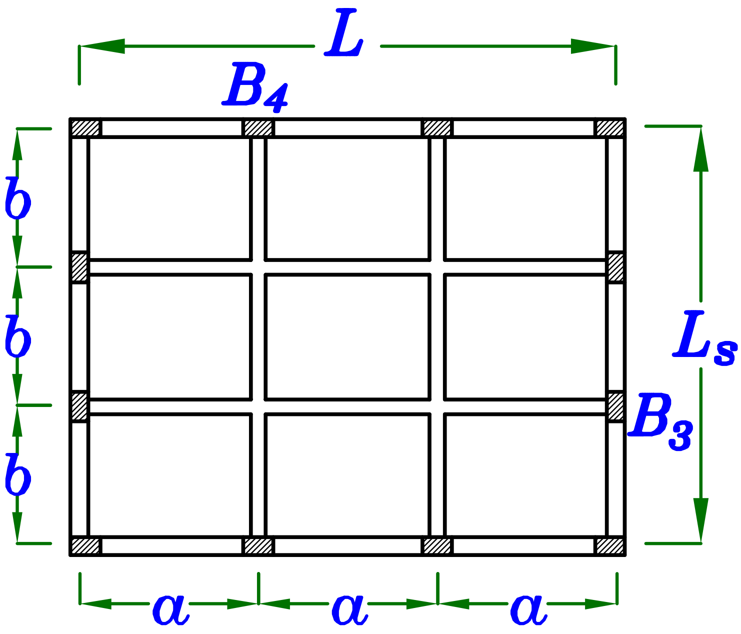
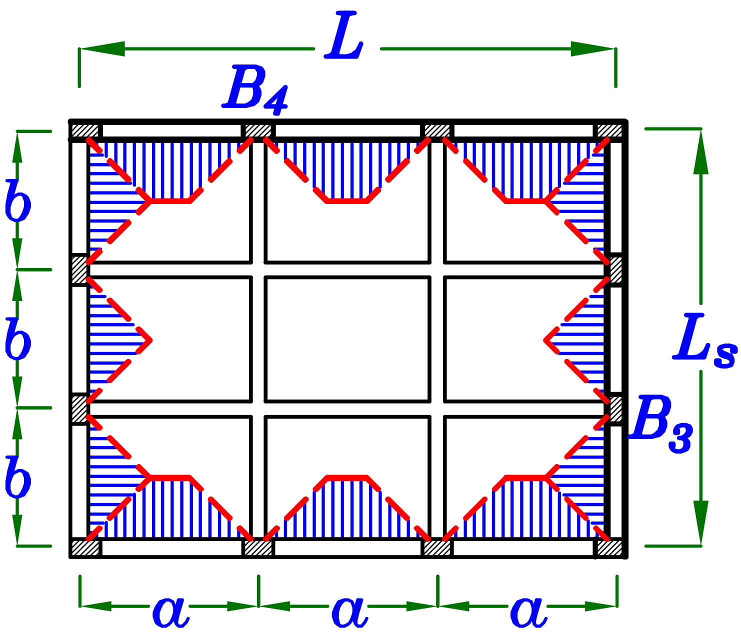
٢- اذا كانت كل الكمرات ال *Panelled* محموله على الاعمده .



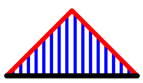
أى ان وزن الكمرات ال *Panelled* يذهب مباشره على الاعمده و لن يحمل على الكمرات الخارجيه .

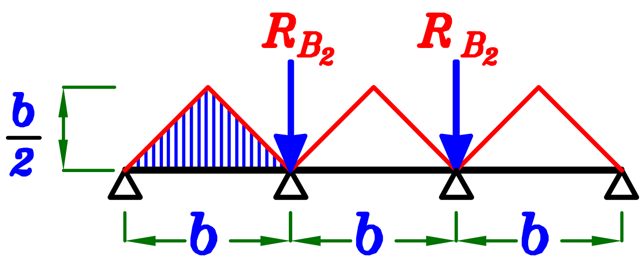
لذا لن نستطيع ان نحسب الاحمال على الكمرات الخارجيه عن طريق w_{av}

بل يجب ان نستخدم w_s

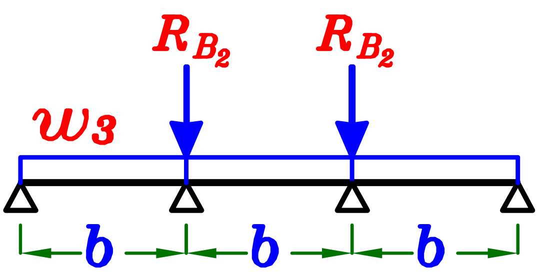


<u>B_3</u>





$$w_3 = (o.w.)_{beam} + \frac{C_a}{C_e}(w_s)\left(\frac{b}{2}\right)$$



Solve it using Empirical Values

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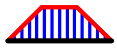
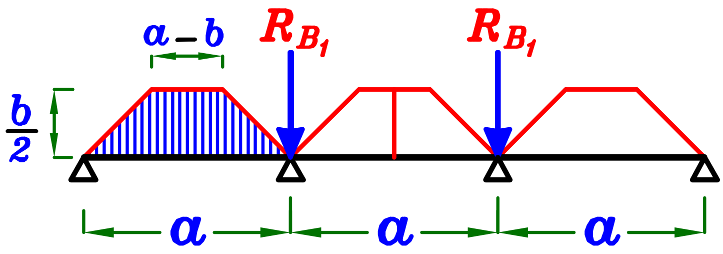
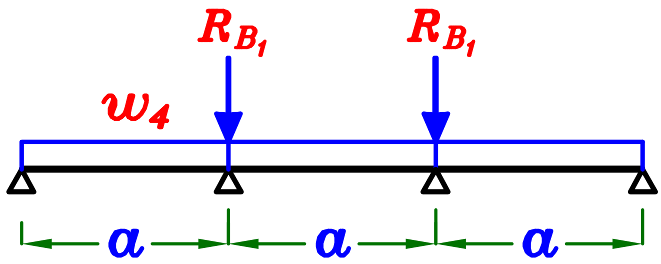
B_4



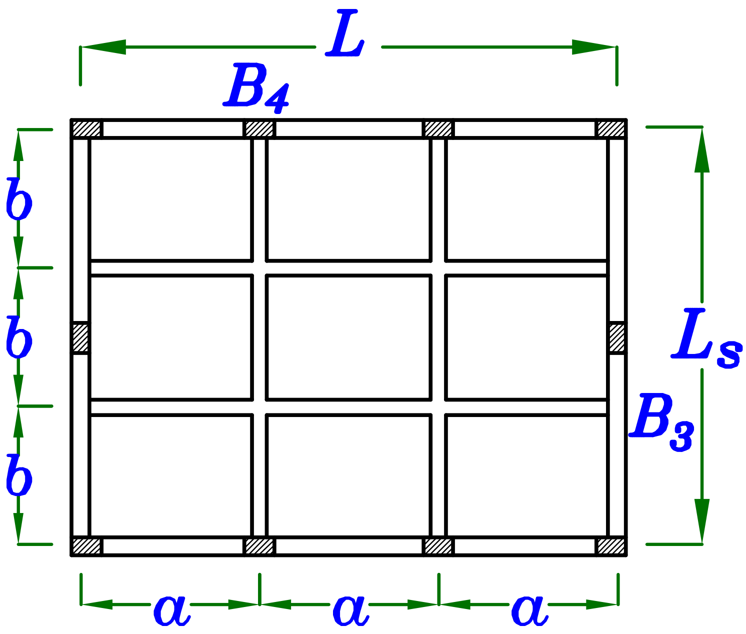
$$w_4 = (o.w.)_{beam} + \frac{C_a}{C_e}(w_s)\left(\frac{b}{2}\right)$$



Solve it using Empirical Values



Special Case.



اذا وجد اتجاه الكمرات ال panelled
محموله على الكمرات الخارجيه
و الاتجاه الاخر محموله على اعمده .

ممكن للتسهيل عند حساب الاحمال
اتجاه نحسبه بالطريقه ال approximate
أى بأستخدام w_{av} لحساب الوزن .

و الاتجاه الاخر بالطريقه ال exact
أى بأستخدام w_s لحساب الوزن .

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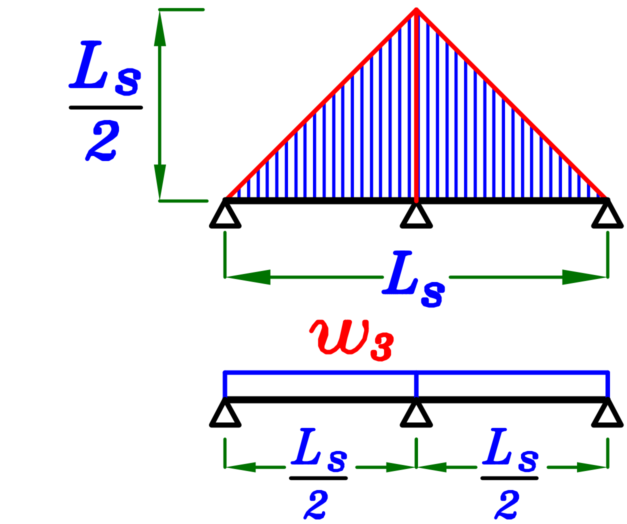
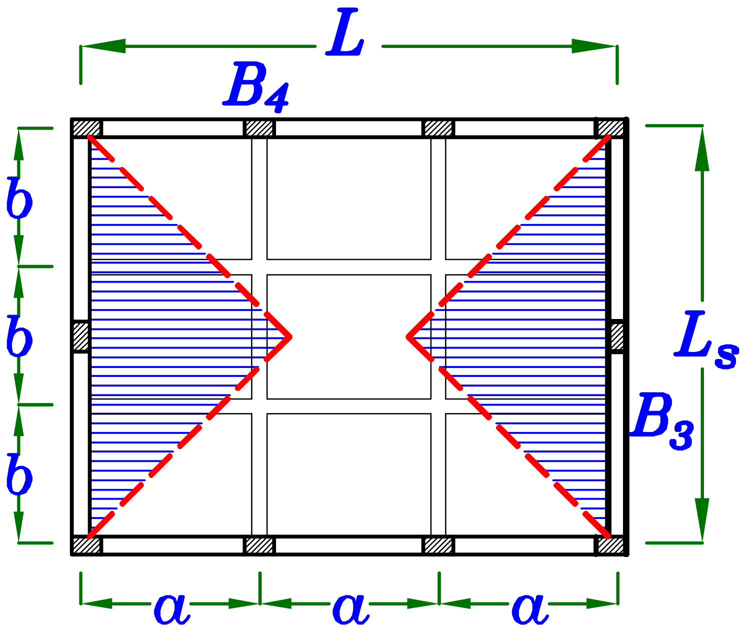
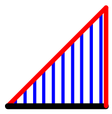
B_3

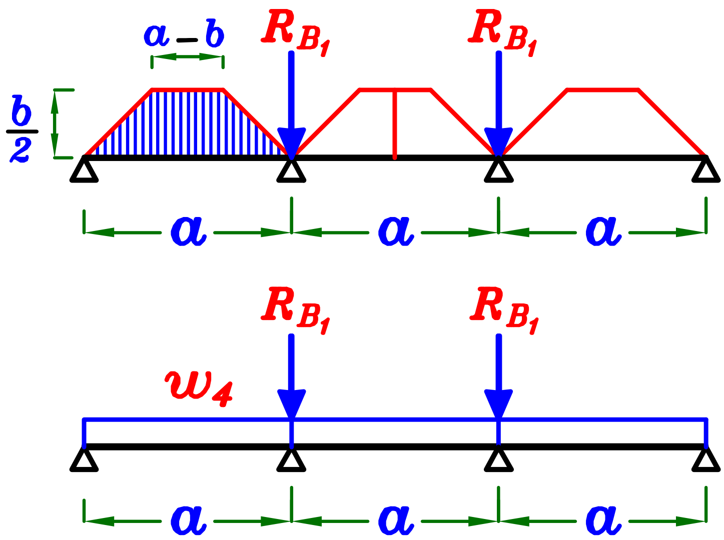


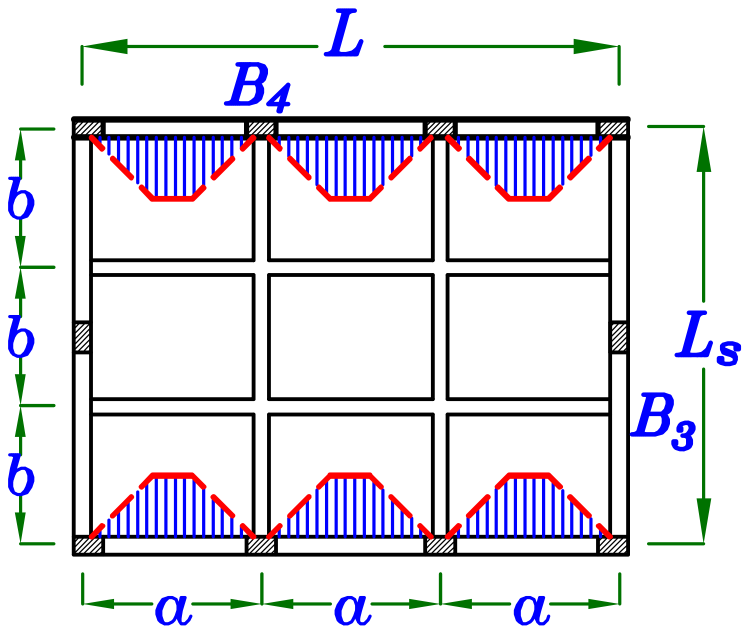


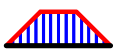


$$w_3 = (o.w.)_{beam} + \frac{\sum area}{span}(w_{av.})$$

B_4







$$w_4 = (o.w.)_{beam} + \frac{C_a}{C_e}(w_s)\left(\frac{b}{2}\right)$$

Solve it using Empirical Values

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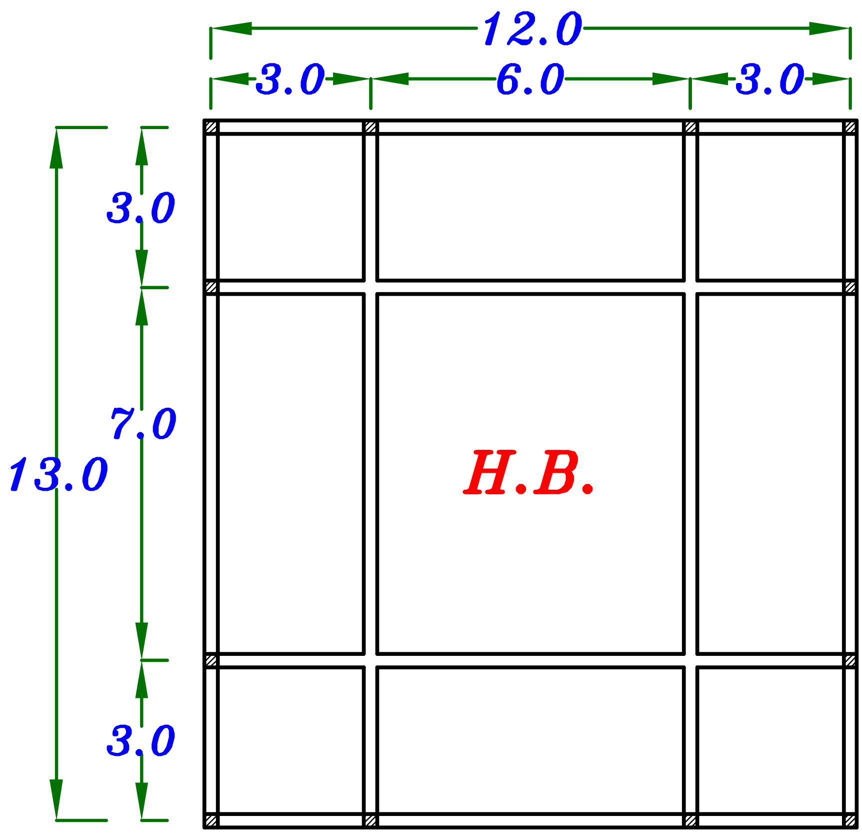
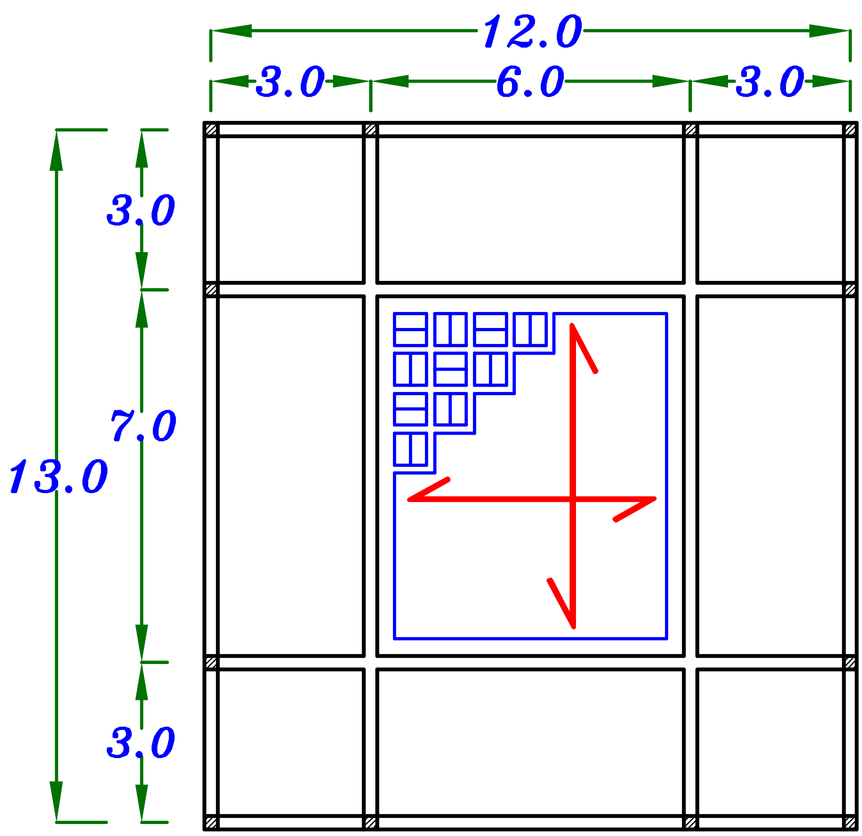
Note.

اذا وجدت بلاطات **Hollow Blocks** محموله على **Panelled Beams**
يجب أن تكون البلاطه الوسطى **Two way Hollow Blocks**
حتى يتوزع الحمل على الكمرات الـ **Panelled** تقريبا بالتساوى





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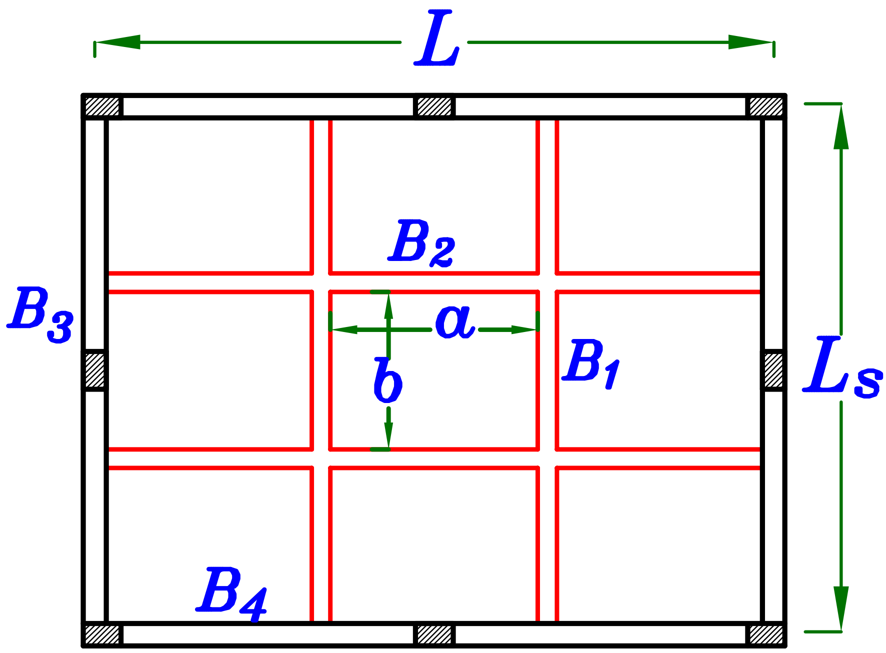
Panelled Beams General Examples.



Steps of Design.

① Design the slabs. (Two way slab يفضل أن تكون)

(Solid or Hollow Blocks Slabs) $(a * b)$

a- Choose the Thickness of the Slab (t_s).

b- Get the Loads on the Slab (w_s).

c- Get the Load Factors α, β.

d- Take a strips in the slab (at the Load direction)

And then Get (B.M.) on the Slab & Design the slab.

e- Draw Details of RFT. of the slab in plan.

② Design the Panelled Beams. (B_1, B_2)

a- Get the Dimensions of the beam. (b, t).

b- Get the avarage Load on the Slab. (w_{av}).

c- Calculate α, β (For the hall area) By using Grashoff.

d- Get the Loads on the Panelled Beam & Calculate the B.M.

e- Calculate the reduction Factor of the B.M. $\left(\frac{\sin \theta}{\sin 90}\right)$

F- Design the Panelled Beam.

g- Draw Details of RFT. For the Panelled Beams.

③ Design the Edge Beams.

(The Exterior Beam) (B_3, B_4)

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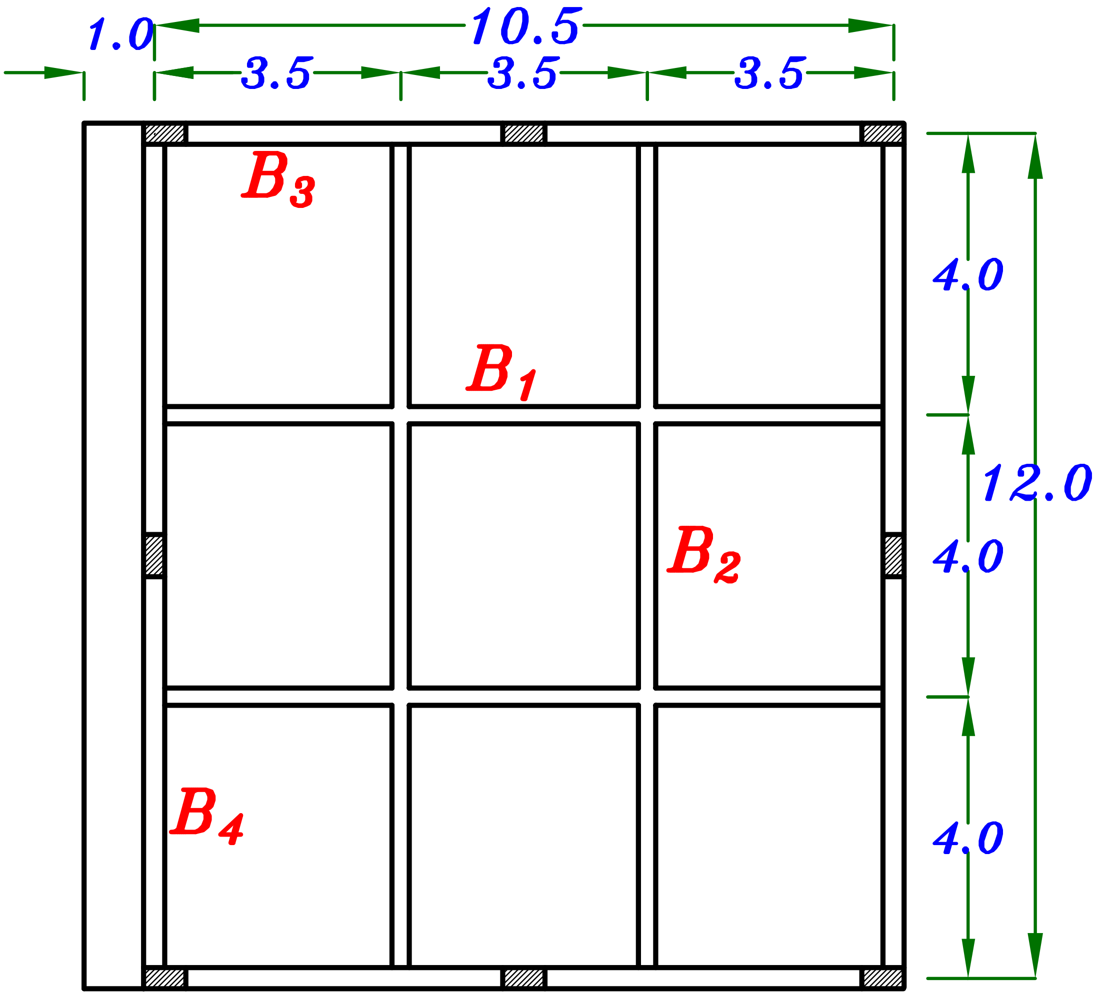
Example.



Data.

$F_{cu} = 25 \ N\backslash mm^2$ $\qquad$ $F_y = 360 \ N\backslash mm^2$

$F.C. = 1.5 \ kN\backslash m^2$ $\qquad$ $L.L. = 4.0 \ kN\backslash m^2$

Req.

1. Design the Slabs as Solid Slabs.
2. Draw Details of RFT. of Slabs in plan.
3. Design Beams B_1, B_2, B_3, B_4
4. Draw Details of RFT. of the beams in elevation & cross sections.

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① Design the Slabs as Solid Slabs.

a- Choose the Thickness of the Slab. (t_s).

S_1 two way $L_s = 3.5\ m$

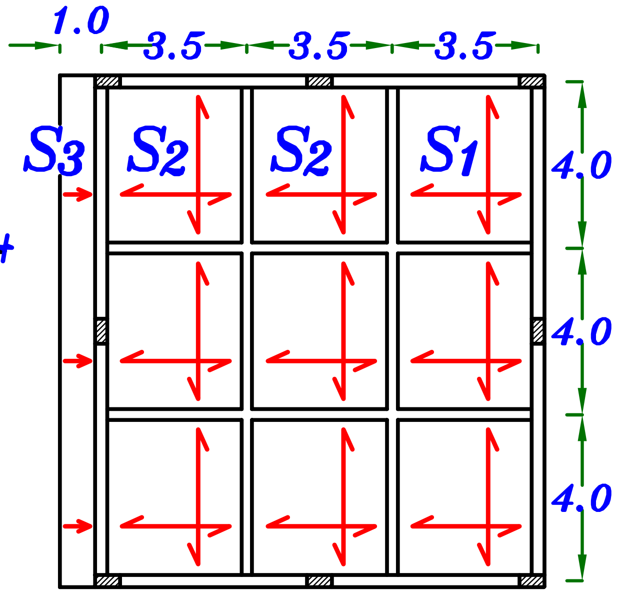
$$t_s = \frac{3500}{40} = 87.5\ mm$$

S_2 two way $L_s = 3.5\ m$

$$t_s = \frac{3500}{45} = 77.7\ mm$$

S_3 Cantilever $L_c = 1.0\ m$

$$t_s = \frac{1000}{10} = 100\ mm$$



Take (t_s) the bigger value $\boxed{t_s = 100\ mm}$

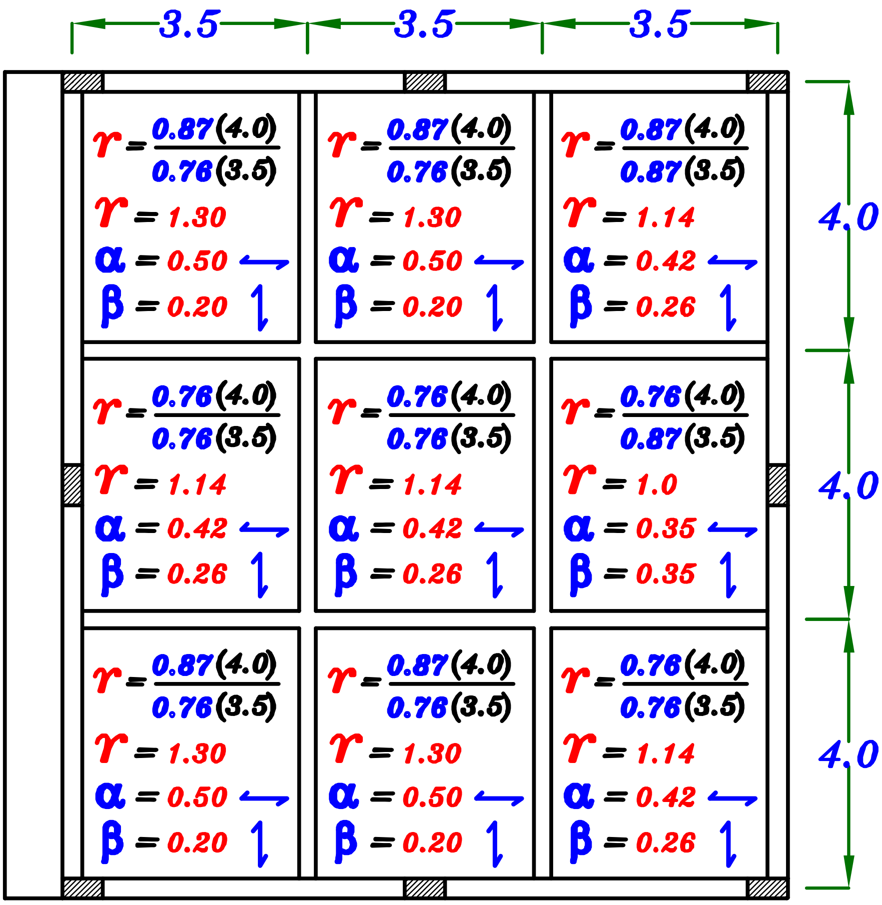
b- Get the Loads on the Slab (w_s).

$$w_s = 1.4\,(0.10 * 25 + 1.50) + 1.6\,(4.0) = 12.0\ kN\backslash m^2$$

c- Get the Load Factors α, β

$$\boxed{\alpha = 0.5\,\gamma - 0.15}$$

$$\boxed{\beta = \frac{0.35}{\gamma^2}}$$



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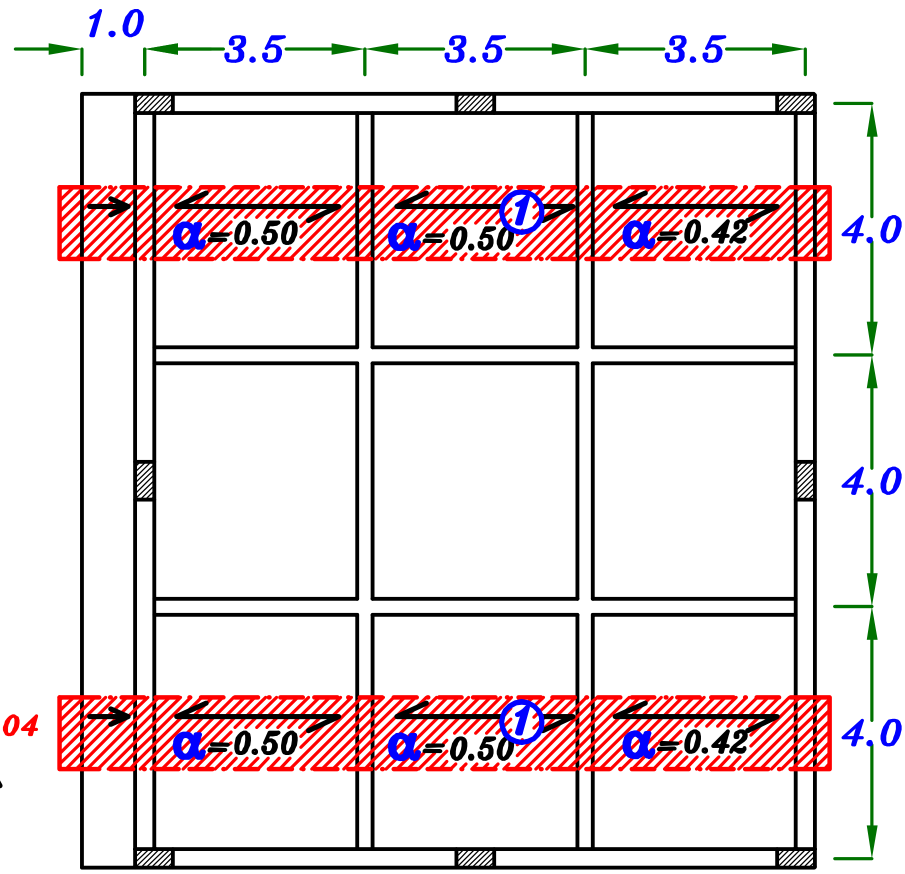
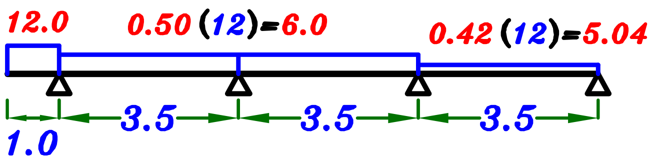
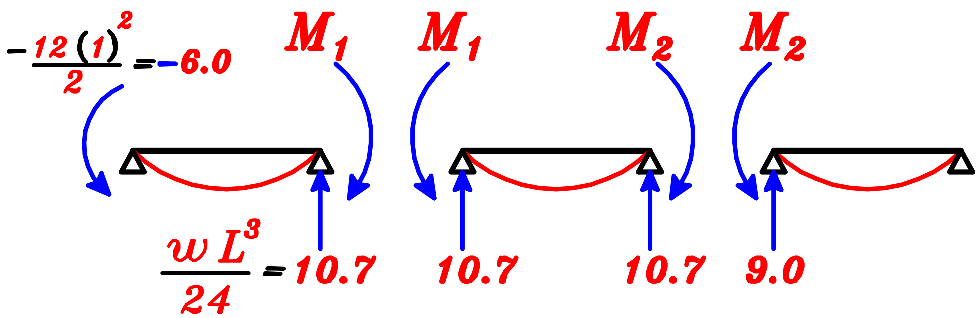
d- Take a strips in the slab (at the Load direction)

And then Get (B.M.) on the Slab & Design the slab.



Strip ①





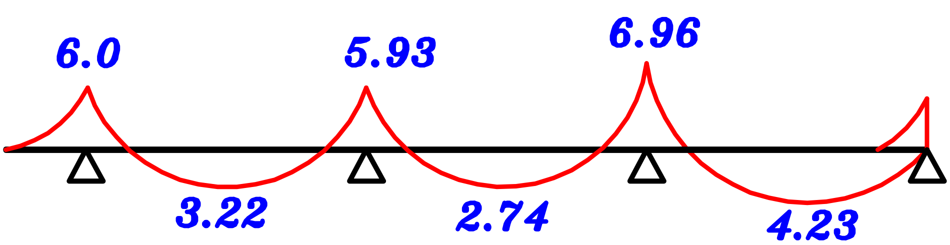
$$-6(3.5)+2M_1(3.5+3.5)+M_2(3.5)=-6(10.7+10.7)$$

$$14M_1+3.5M_2=-107.4 \quad ----①$$

$$M_1(3.5)+2M_2(3.5+3.5)+0.0=-6(10.7+9.0)$$

$$3.5M_1+14M_2=-118.2 \quad ----②$$

$$\boxed{M_1=-5.93\ kN.m} \qquad \boxed{M_2=-6.96\ kN.m}$$



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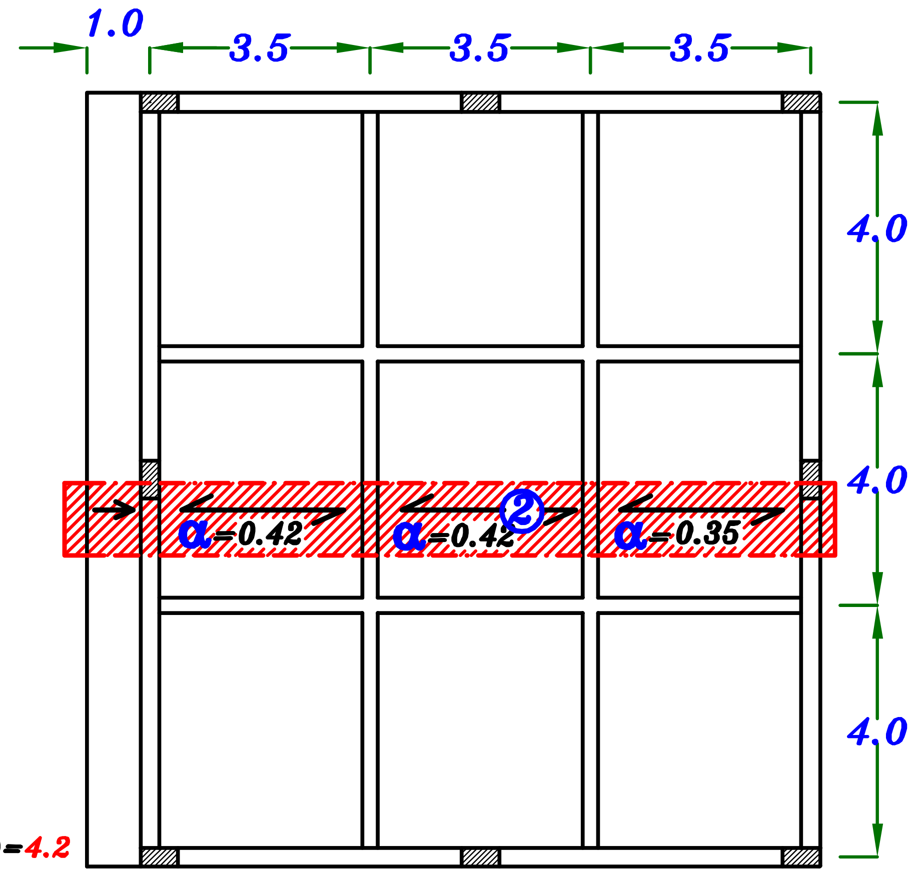
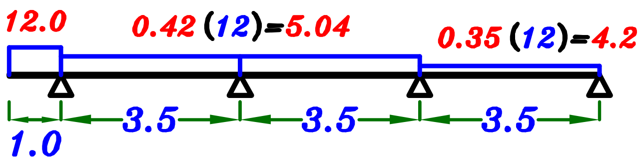
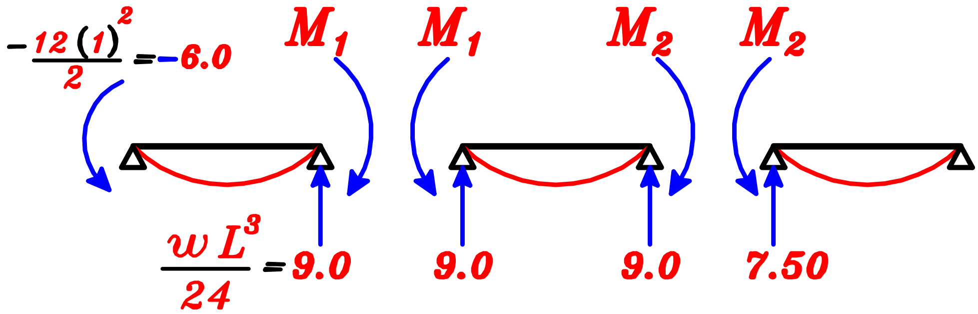
Strip ②







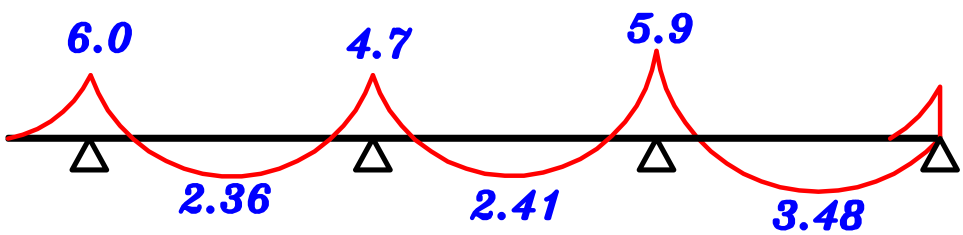
$$-6(3.5) + 2M_1(3.5+3.5) + M_2(3.5) = -6\ (9.0+9.0\ )$$

$$14\ M_1 + 3.5\ M_2 = -87.0 \qquad ----1$$

$$M_1(3.5) + 2M_2(3.5+3.5) + 0.0 = -6\ (9.0 + 7.5)$$

$$3.5\ M_1 + 14\ M_2 = -99.0 \qquad ----2$$

$\boxed{M_1 = -4.70 \ \ kN.m}$ $\boxed{M_2 = -5.90\ kN.m}$



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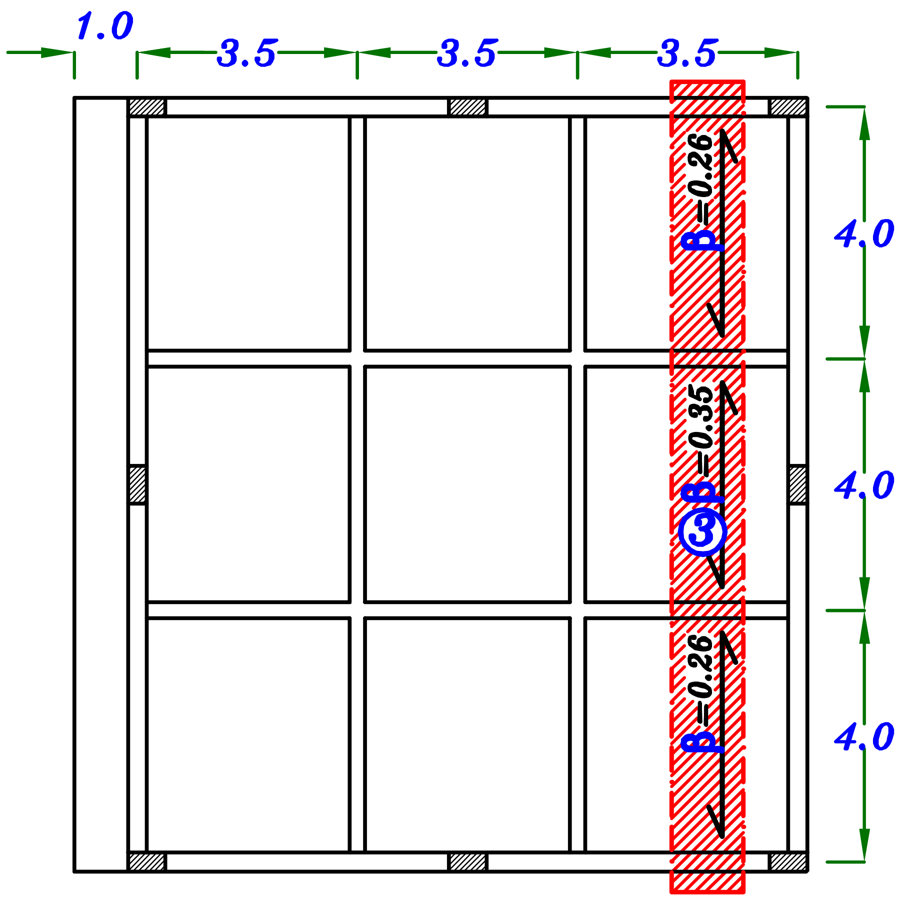
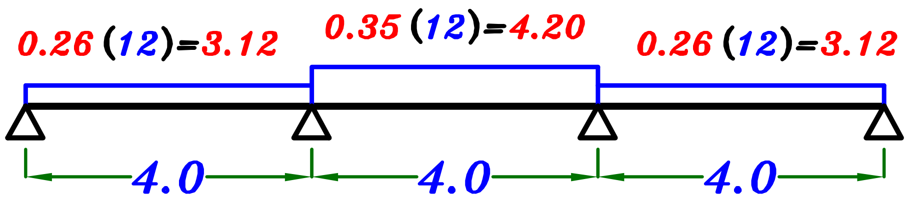
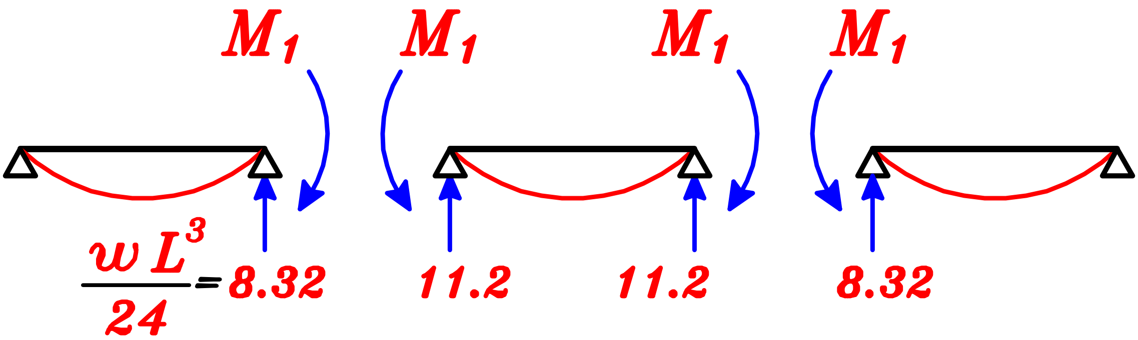
Strip ③

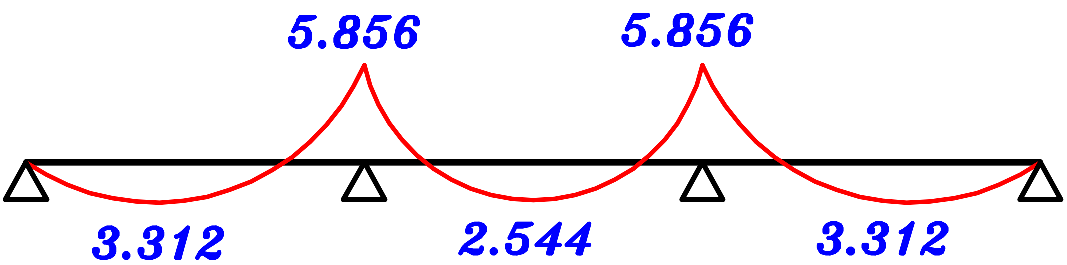






$$0.0 + 2M_1(4.0+4.0) + M_1(4.0) = -6\,(8.32 + 11.2)$$

$$\boxed{M_1 = -5.856 \ \ kN.m}$$



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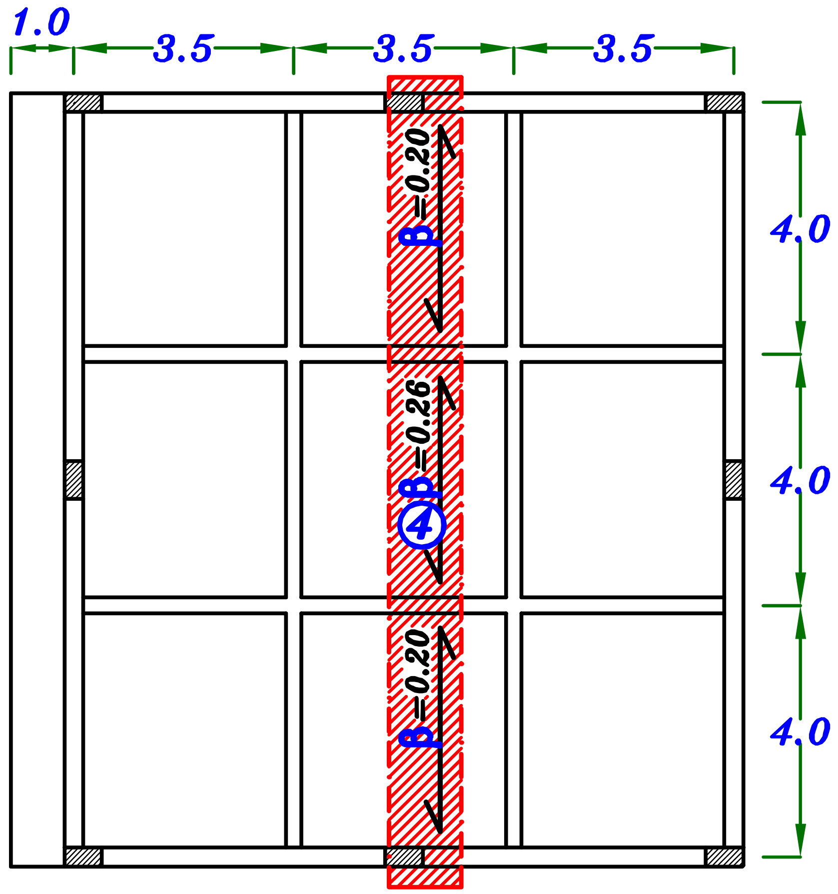
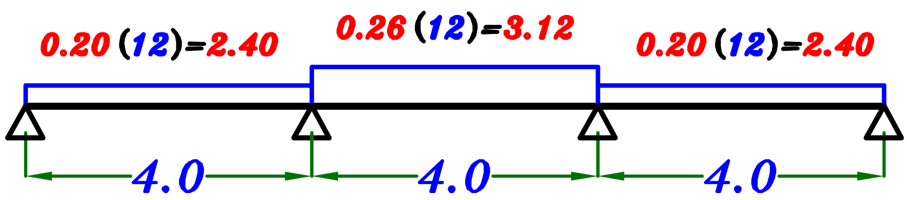
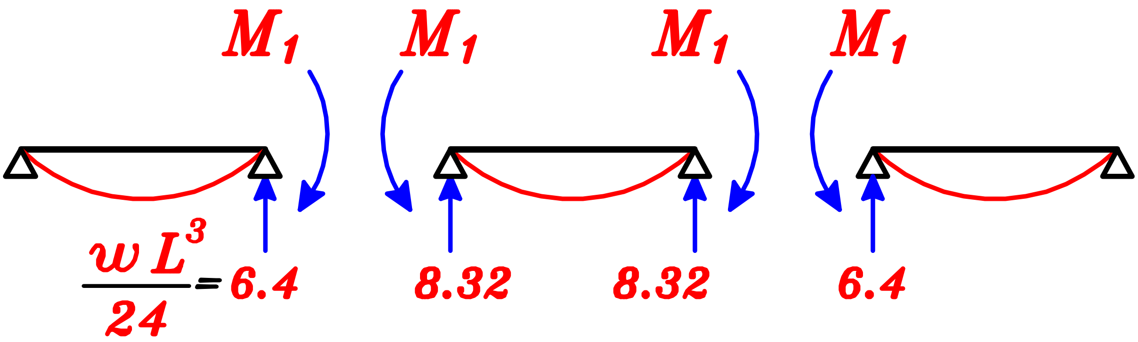
Strip 4

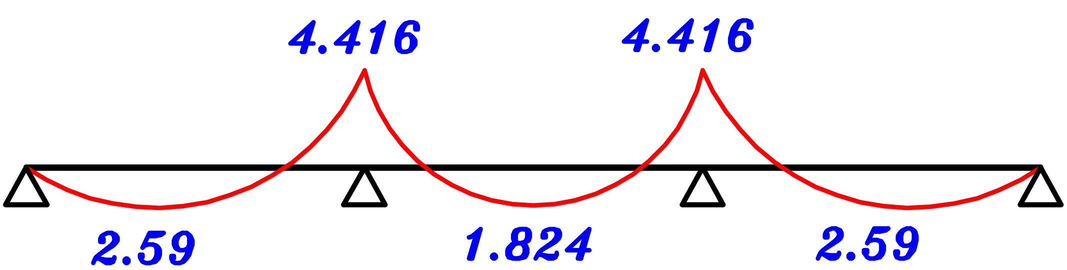






$$0.0 + 2M_1(4.0+4.0) + M_1(4.0) = -6(6.4 + 8.32)$$

$$\boxed{M_1 = -4.416 \ kN.m}$$



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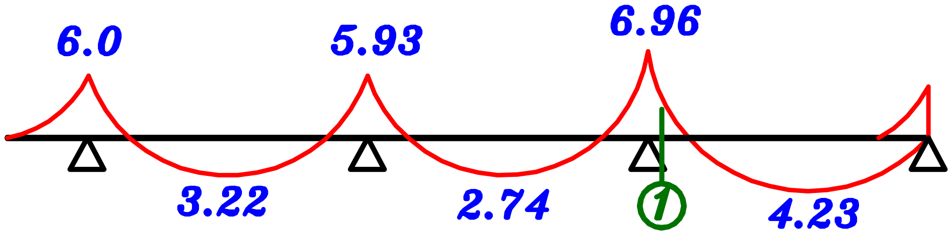
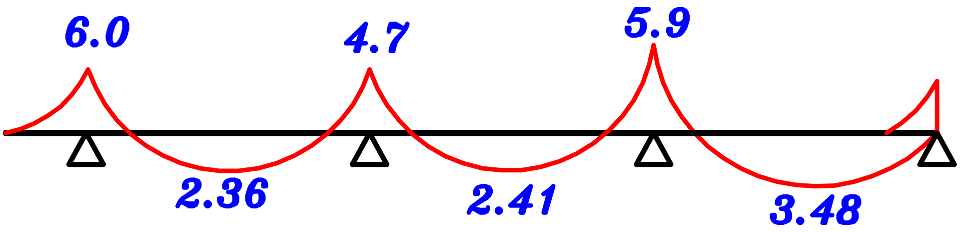
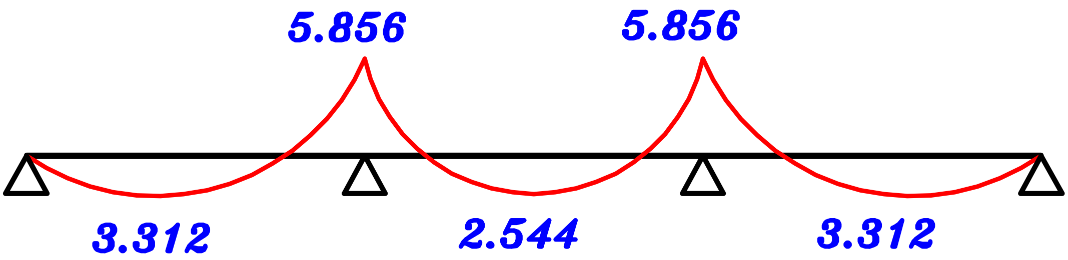
Strip 1



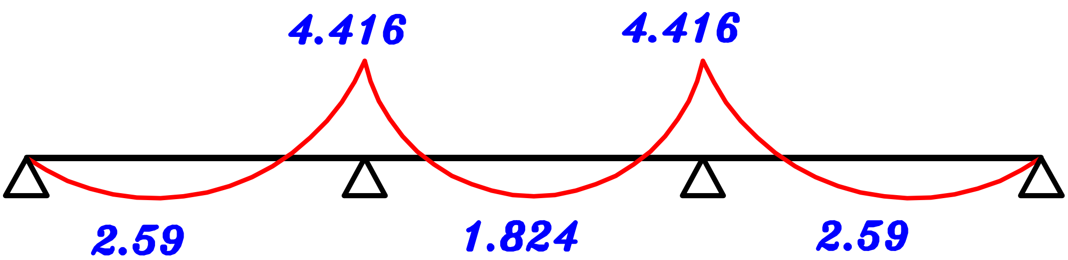
Strip 2



Strip 3



Strip 4



Sec. 1

$M_{U.L.} = 6.96 \;\; kN.m\backslash m$

$t_s = 100\ mm$ ، $d = 100 - 20 = 80\ mm$ ، $B = 1000\ mm$ عرض الشريحه

$$80 = C_1 \sqrt{\frac{6.96 * 10^6}{25 * 1000}} \longrightarrow C_1 = 4.79 \longrightarrow J = 0.825$$

$$A_s = \frac{6.96 * 10^6}{0.825 * 360 * 80} = 293 \;\; mm^2/m$$

$\boxed{5\phi 10 \backslash m}$

∴ سيؤخذ تسليح باقى القطاعات $5\phi 10\backslash m$

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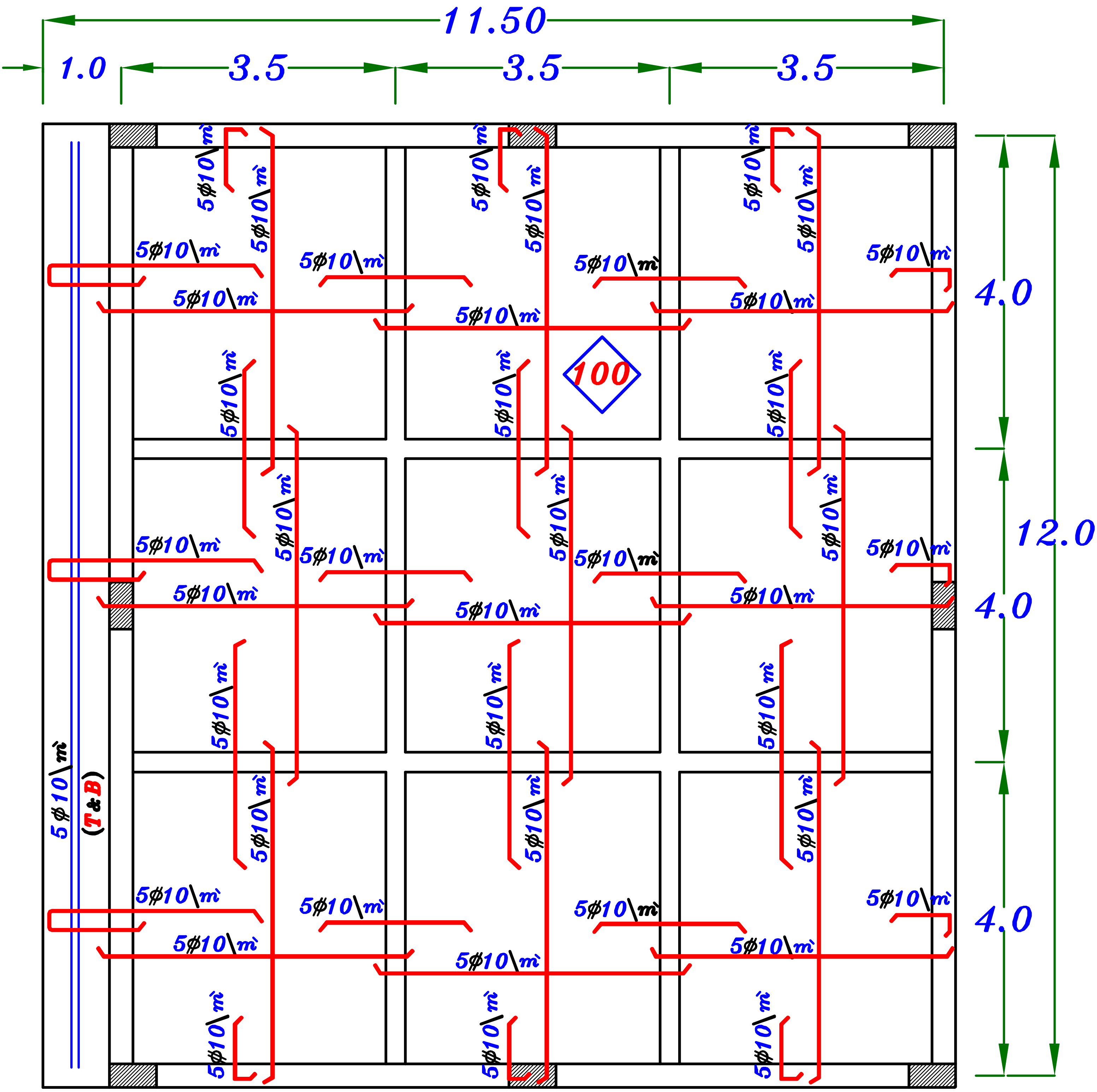
Details of RFT. For the Slab.



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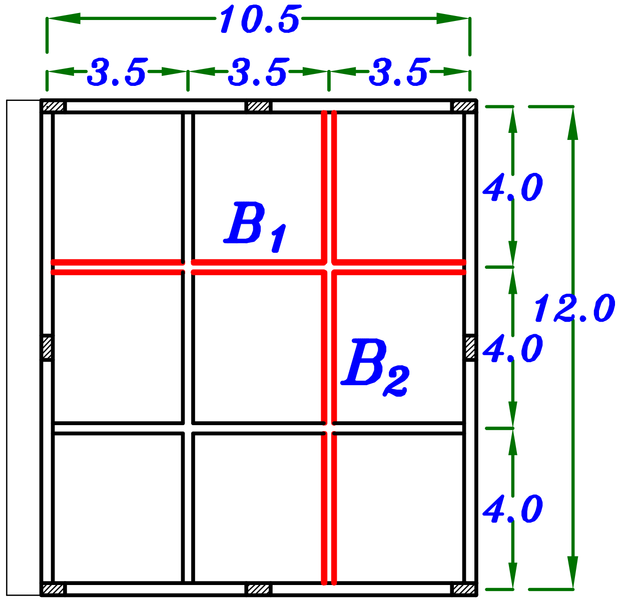
② Design of Panelled Beams.

a – Get the Dimensions of the beam. (b, t)



Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{10.5}{16} = 0.656\ m$$

$$= 0.70\ m$$

$\boxed{b = 0.25\ m}$ $\boxed{t = 0.70\ m}$

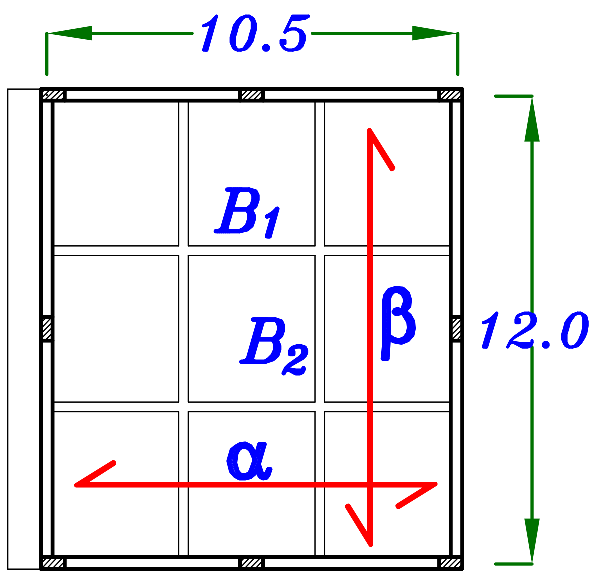
b – Get the Loads on the Slab. (w_{av})

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s}$$

$$w_{av.} = w_s + \frac{1.4 * b\,(t - t_s)\,[\text{مجموع أطوال الكمرات الداخلية}] * \gamma_c}{L * L_s}$$

$$w_{av.} = 12.0 + \frac{1.4 * 0.25\,(0.7 - 0.1)\,[2 * 12 + 2 * 10.5] * 25}{12 * 10.5} = 13.87\ kN\backslash m^2$$

c – Calculate α, β By using Grashoff.



$$r = \frac{m\ L}{m'\ L_s} = \frac{(1.0)\ 12.0}{(1.0)\ 10.5} = 1.142$$

$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.142)^4}{1 + (1.142)^4} = 0.63$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.142)^4} = 0.37$$

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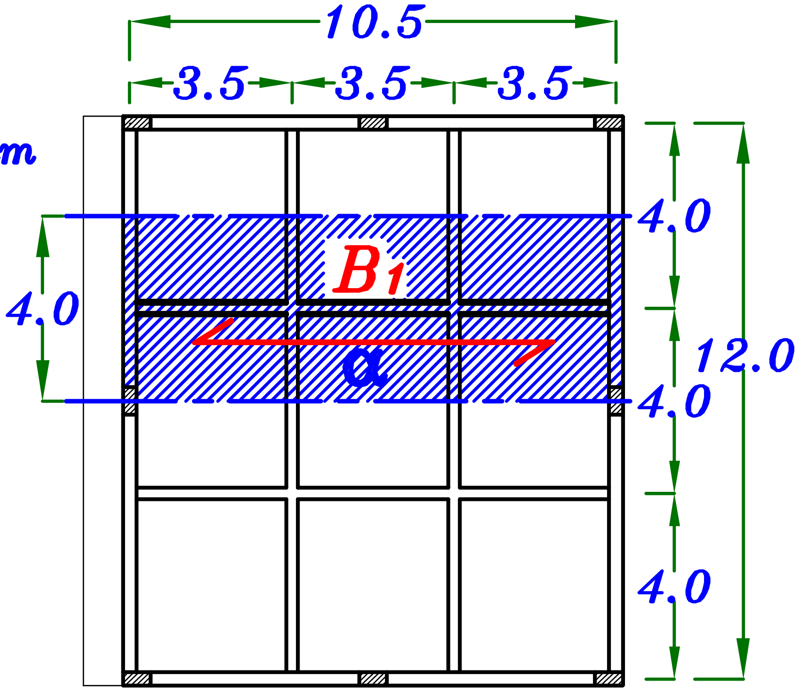
B_1 α Direction.



d- Get the Loads on the Panelled Beam & Calculate the B.M.

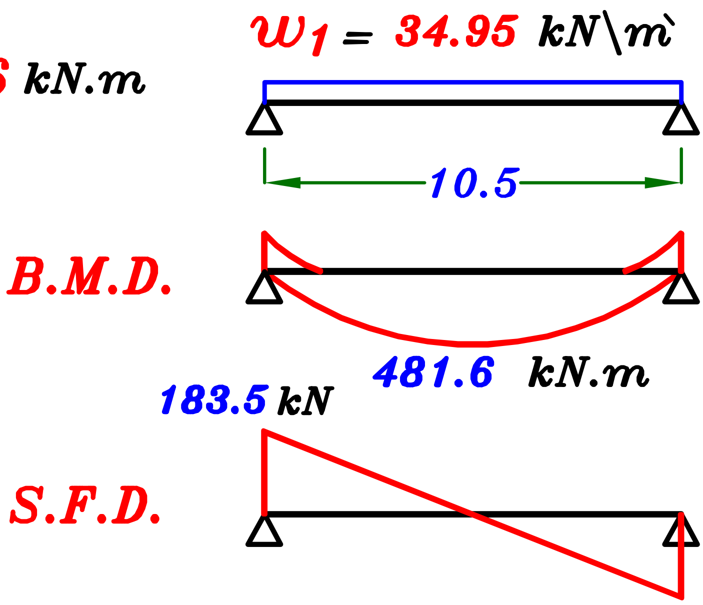
$$a = 4.0\ m$$

$$w_1 = w_{av.} * a * \alpha$$

$$= 13.87 * 4.0 * 0.63$$

$$= 34.95\ kN\backslash m`$$

$$M = 34.95 * \frac{10.5^2}{8} = 481.6\ kN.m$$



e - Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 4.0\ m \quad , \quad \frac{L}{2} = 6.0\ m$$

$$\theta_{B_1} = \frac{4.0}{6.0} * 90^\circ = 60^\circ$$

$$M_1 = 481.6 * \frac{\sin 60^\circ}{\sin 90^\circ} = 417.0\ kN.m$$

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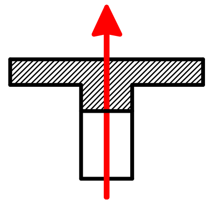
F- Design the Panelled Beam. B_1

α Direction. $\therefore$ Cover = 50 mm

$t = 700\ mm$ $d = 700-50 = 650\ mm$



$$B = \begin{cases} C.L.-C.L. = 4.0\ m = 4000\ mm \\ 16\,t_s + b = 16 * 100 + 250 = 1850\ mm \\ K\dfrac{L}{5} + b = 1.0 * \dfrac{10500}{5} + 250 = 2350\ mm \end{cases}$$

$$\boxed{B = 1850\ mm}$$

$$650 = C_1\sqrt{\frac{417.0 * 10^6}{25 * 1850}} \longrightarrow C_1 = 6.84 \longrightarrow J = 0.826$$

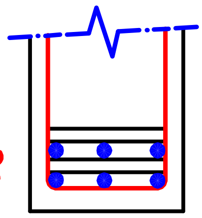
$$A_s = \frac{417.0 * 10^6}{0.826 * 360 * 650} = 2157\ mm^2$$

Check $A_{s_{min.}}$ $A_{s_{req.}} = 2157\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d$ $\therefore$ Take $A_s = A_{s_{req.}} = 2157\ mm^2$ $6\phi22$

$$n = \frac{b-25}{\phi+25} = \frac{250-25}{22+25} = 4.78 = 4.0\ bars$$



Stirrup Hangers = $(0.1 \rightarrow 0.2)\,A_s = (215 \rightarrow 431\ mm^2)$ $3\phi12$

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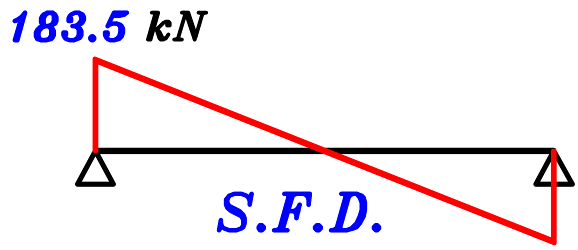
Check Shear.



$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \quad N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \quad N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\,d} = \frac{183.5 * 10^3}{250 * 650} = 1.13 \quad N\backslash mm^2 \qquad \therefore q_{cu} < q_s < q_{u_{max}}$$

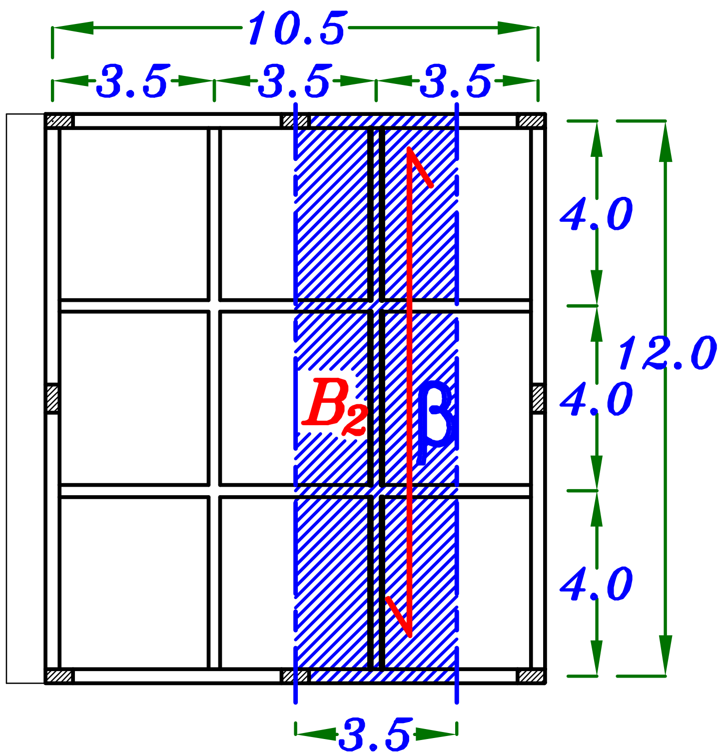
$$q_s - \frac{q_{cu}}{2} = \frac{n\,A_s(F_y\backslash \gamma_s)}{b\;S} \xrightarrow{Take} n = 2 \;,\; \phi 8 = 50.3 \;mm^2$$

$$1.13 - \frac{0.98}{2} = \frac{2\,(50.3)\,(240\backslash 1.15)}{250 * S} \longrightarrow S = 131.2 \quad mm$$

$$No.\ of\ stirrups\backslash m' = \frac{1000}{S} = \frac{1000}{131.2} = 7.62 \quad Use \quad 8\,\phi 8\backslash m'$$

B_2 β Direction.

d– Get the Loads on the Panelled Beam & Calculate the B.M.



$$b = 3.5\,m$$

$$w_2 = w_{av.} * b * \beta$$

$$= 13.87 * 3.5 * 0.37$$

$$= 17.96 \;kN\backslash m'$$

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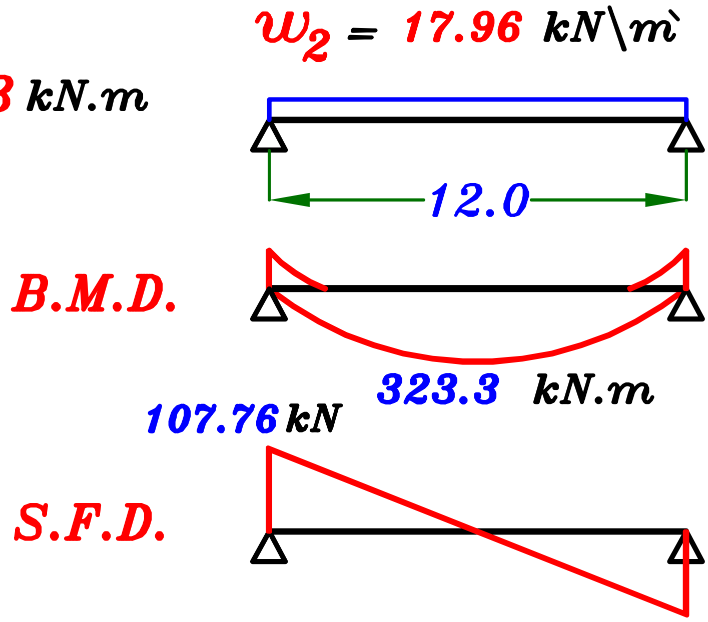
$$M = 17.96 * \frac{12.0^2}{8} = 323.3 \ kN.m$$



e- Calculate the reduction Factor of the B.M. $\left(\frac{\sin \theta}{\sin 90}\right)$

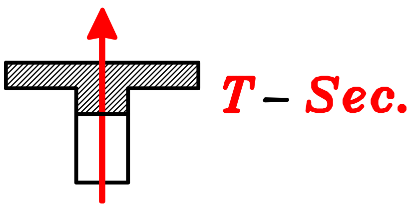
$$X = 3.5 \ m \quad , \quad \frac{L}{2} = 5.25 \ m$$

$$\theta_{B_1} = \frac{3.5}{5.25} * 90^\circ = 60^\circ$$

$$M_1 = 323.3 * \frac{\sin 60^\circ}{\sin 90^\circ} = 279.98 \ kN.m$$

F- Design the Panelled Beam. B_1

β Direction. $\therefore$ Cover = 70 mm



$$t = 700 \ mm \qquad d = 700 - 70 = 630 \ mm$$

$$B = \begin{cases} C.L.-C.L. = 3.5 \ m = 3500 \ mm \\ 16 \ t_s + b = 16 * 100 + 250 = 1850 \ mm \\ K \frac{L}{5} + b = 1.0 * \frac{12000}{5} + 250 = 2650 \ mm \end{cases} \qquad \boxed{B = 1850 \ mm}$$

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$$630 = C_1\sqrt{\frac{279.98 * 10^6}{25 * 1850}} \longrightarrow C_1 = 9.55 \longrightarrow J = 0.826$$

$$A_s = \frac{279.98 * 10^6}{0.826 * 360 * 630} = 1494.5\ mm^2$$

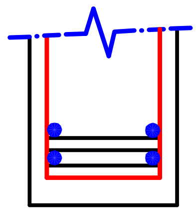
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1494.5\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 630 = 492.2\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1494.5\ mm^2 \quad 4\phi 22$$

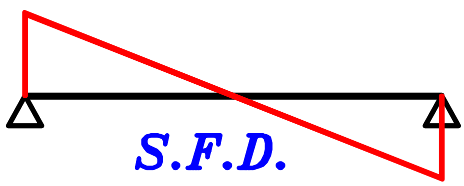
$$n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{22 + 25} = 4.78 = 4.0 \text{ bars}$$

$4\phi 22$



$$\text{Stirrup Hangers} = (0.1 \rightarrow 0.2)\, A_s = (107 \rightarrow 214\ mm^2) \quad 2\phi 12$$

Check Shear.



$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98\ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85\ N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\, d} = \frac{107.76 * 10^3}{250 * 650} = 0.66\ N\backslash mm^2$$

$$\therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $\quad 5\phi 8\backslash m$

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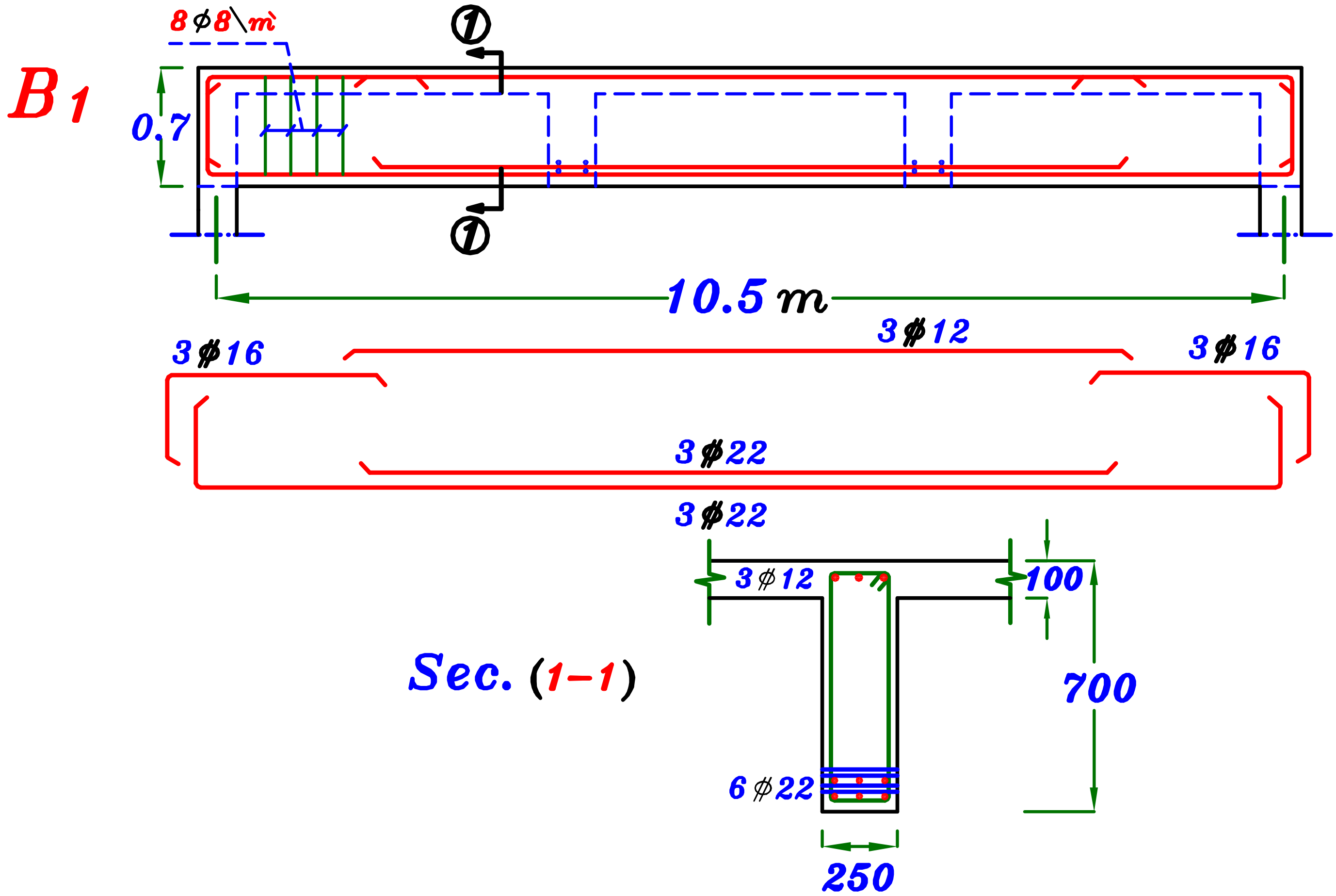
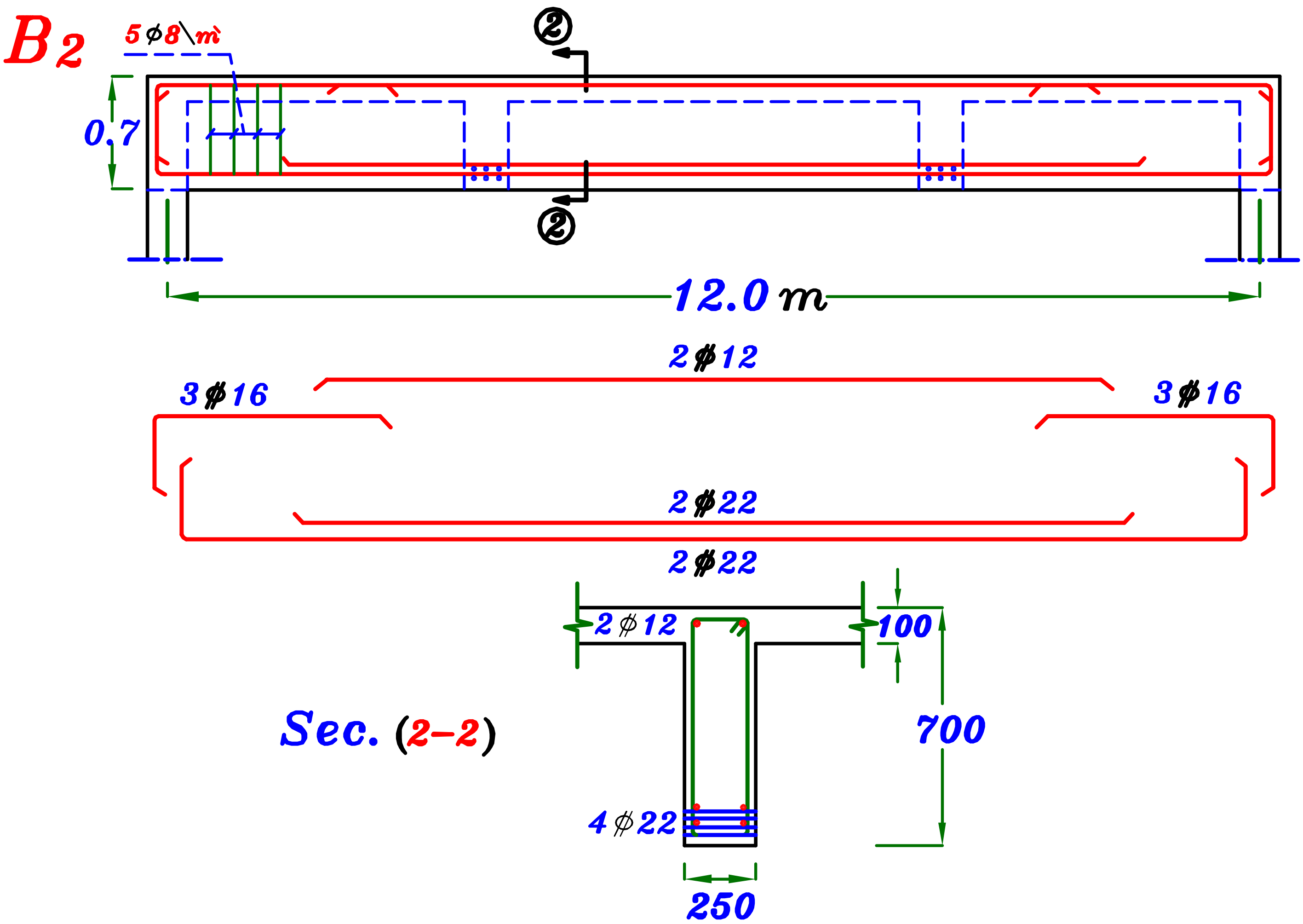
9- Draw Details of RFT. For the Beams (B_1, B_2)





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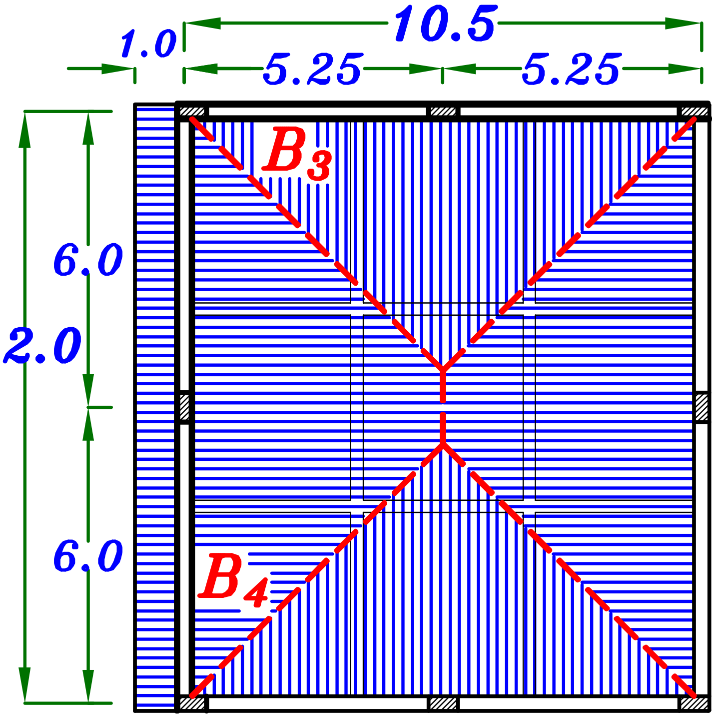
③ Design of Edge Beams. B_3 , B_4

لان الكمرات الـ **Panelled** محموله كلها على الكمرات الخارجيه و ليس الاعمده .
اذا نستطيع حساب الاحمال على الكمرات الخارجيه عن طريق **Approximate Method**

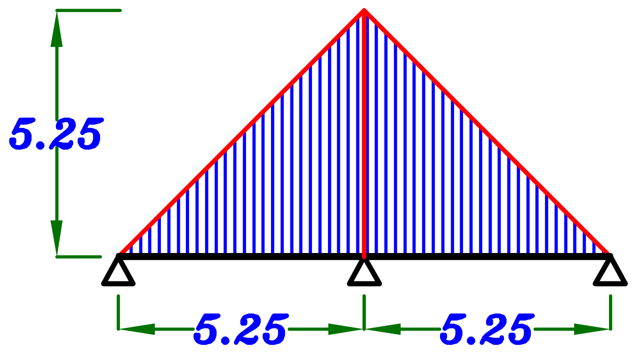


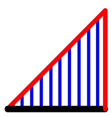
B_3

الـ t للكمره الخارجيه يجب ان لا تقل عن الـ t للكمره الـ **Panelled** المحموله عليها

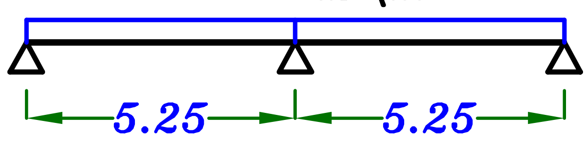
Take $b = 300$ mm , $t = 700$ mm

$$(o.w.)_{U.L.} = 1.4 * b * t * \gamma_c$$
$$= 1.4 * 0.3 * 0.7 * 25$$
$$= 7.35 \ kN\backslash m`$$



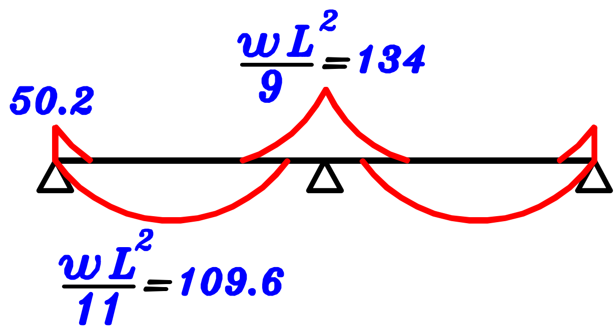


$$\frac{\sum area}{span} = \frac{0.5 * 5.25 * 5.25}{5.25} = 2.625$$



$$w_3 = (o.w.)_{beam} + \frac{\sum area}{span} (w_{av.})$$

$$w_3 = 7.35 + 2.625 * 13.87 = 43.75 \ kN\backslash m`$$



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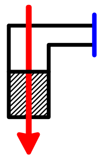
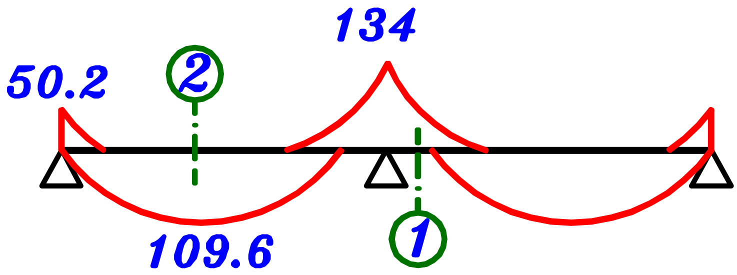
Sec. ① $M_{U.L.} = 134.0\ kN.m$

R–Sec.





$$650 = C_1 \sqrt{\frac{134.0 * 10^6}{25 * 300}} \longrightarrow C_1 = 4.85 \longrightarrow J = 0.826$$

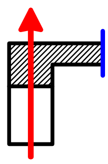
$$A_s = \frac{134.0 * 10^6}{0.826 * 360 * 650} = 693.3\ mm^2$$

Check $As_{min.}$ $\qquad A_{s_{req.}} = 693.3\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore Take\ A_s = A_{s_{req.}} = 693.3\ mm^2 \quad 4\phi 16$$

Sec. ② $M_{U.L.} = 109.6\ kN.m$ **L–Sec.**



$$B = \left\{ \begin{array}{ll} C.L.-C.L. = 6.0\ m = 6000\ mm \\ 6\, t_s + b = 6 * 100 + 300 = 900\ mm \\ K \frac{L}{10} + b = 0.8 * \frac{5250}{10} + 300 = 720\ mm \end{array} \right\} \quad \boxed{B = 720\ mm}$$

$$650 = C_1 \sqrt{\frac{109.6 * 10^6}{25 * 720}} \longrightarrow C_1 = 8.33 \longrightarrow J = 0.826$$

$$A_s = \frac{109.6 * 10^6}{0.826 * 360 * 650} = 567.0\ mm^2$$

Check $As_{min.}$ $\qquad A_{s_{req.}} = 567.0\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore Take\ A_s = A_{s_{req.}} = 567.0\ mm^2 \quad 3\phi 16$$

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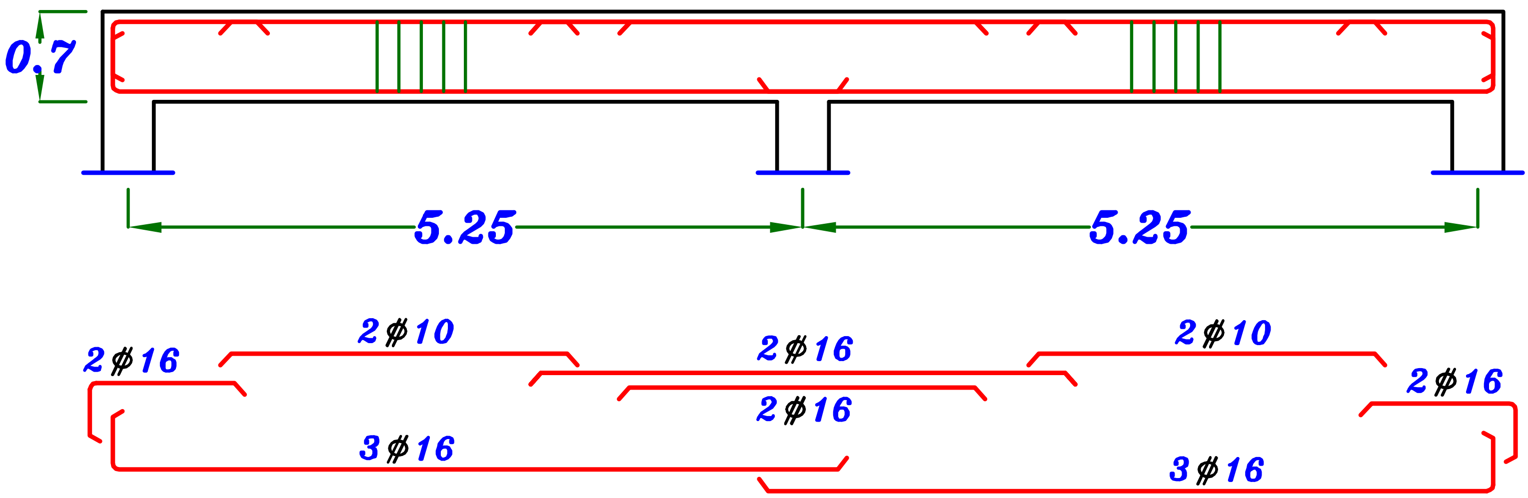
RFT. of Beam B_3

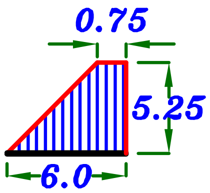


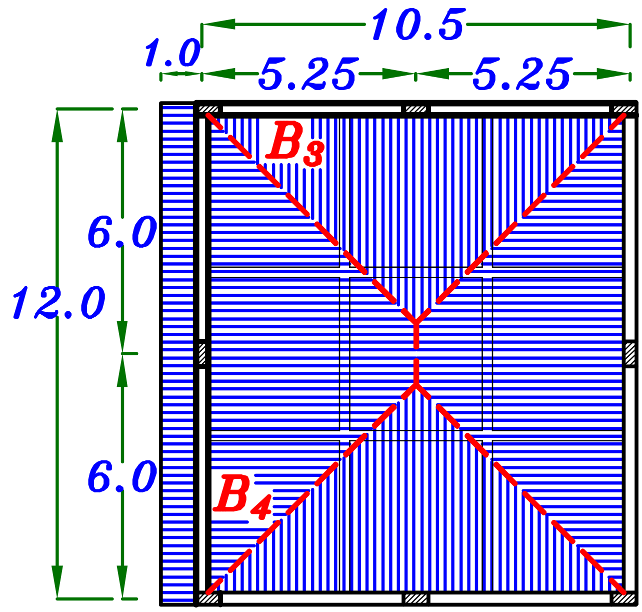
B_4

Take $b = 300$ mm

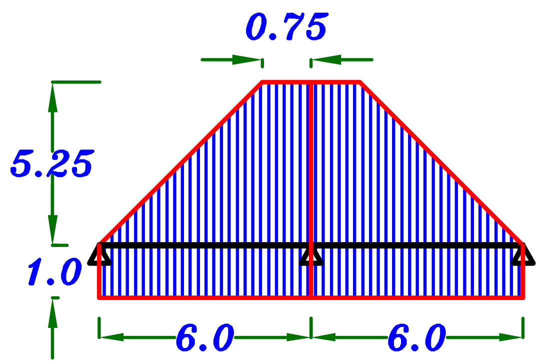
$t = 700$ mm

$(o.w.)_{U.L.} = 7.35$ kN\m`



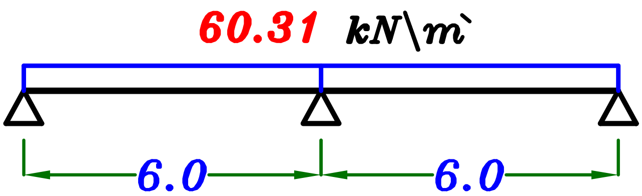


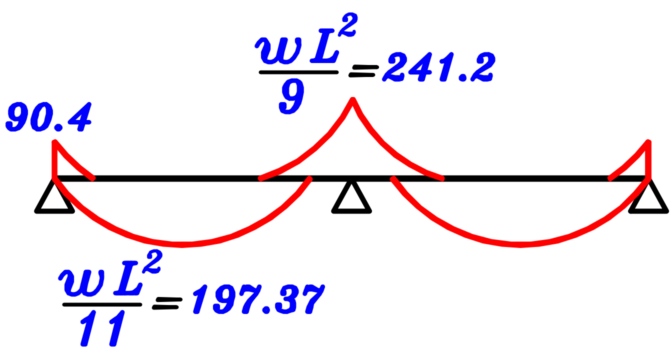
$$\frac{\sum area}{span} = \frac{5.25\left(\frac{6.0+0.75}{2}\right)}{6.0} = 2.953$$



$$w_3 = (o.w.)_{beam} + \frac{\sum area}{span}(w_{av.}) + w_s\, L_c$$

$$w_3 = 7.35 + 2.953 * 13.87 + 12.0 * 1.0 = 60.31 \text{ kN\m`}$$





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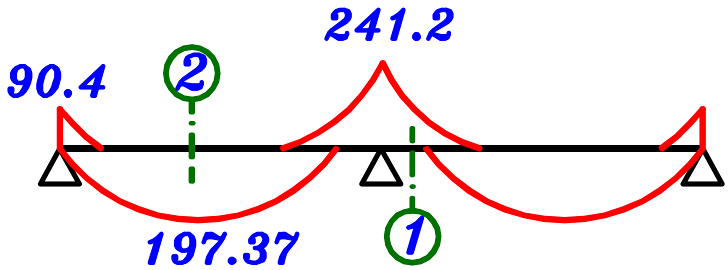
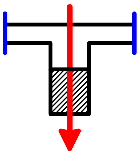
Sec. ① $M_{U.L.} = 241.2\ kN.m$



R-Sec.



$$650 = C_1 \sqrt{\frac{241.2 * 10^6}{25 * 300}} \longrightarrow C_1 = 3.62 \longrightarrow J = 0.786$$

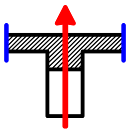
$$A_s = \frac{241.2 * 10^6}{0.786 * 360 * 650} = 1311\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1311\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 300 * 650 = 609.3\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1311\ mm^2 \quad 4\phi22$$

Sec. ② $M_{U.L.} = 197.37\ kN.m$ T-Sec.



$$B = \begin{cases} C.L.-C.L. = 1.0 + 5.25 = 6.25 = 6250\ mm \\ 16\,t_s + b = 16 * 100 + 300 = 1900\ mm \\ K\,\frac{L}{5} + b = 0.8 * \frac{6000}{5} + 300 = 1260\ mm \end{cases} \qquad \boxed{B = 1260\ mm}$$

$$650 = C_1 \sqrt{\frac{197.37 * 10^6}{25 * 1260}} \longrightarrow C_1 = 8.22 \longrightarrow J = 0.826$$

$$A_s = \frac{197.37 * 10^6}{0.826 * 360 * 650} = 1021\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1021\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 300 * 650 = 609.3\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1021\ mm^2 \quad 3\phi22$$

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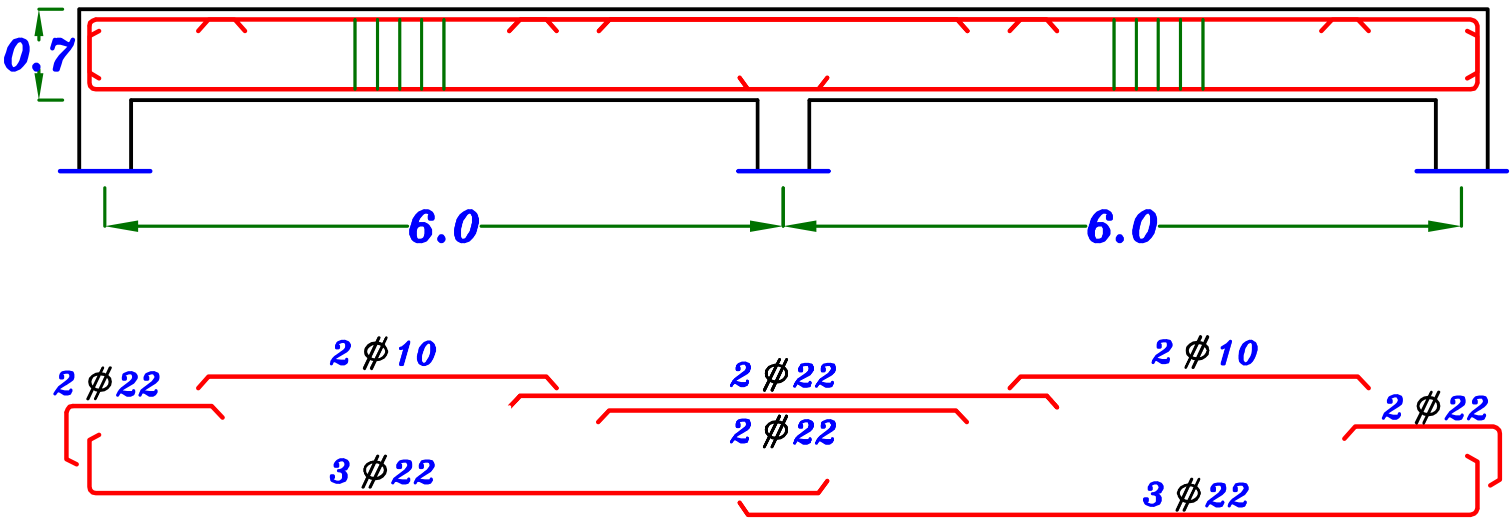
RFT. of Beam B_4



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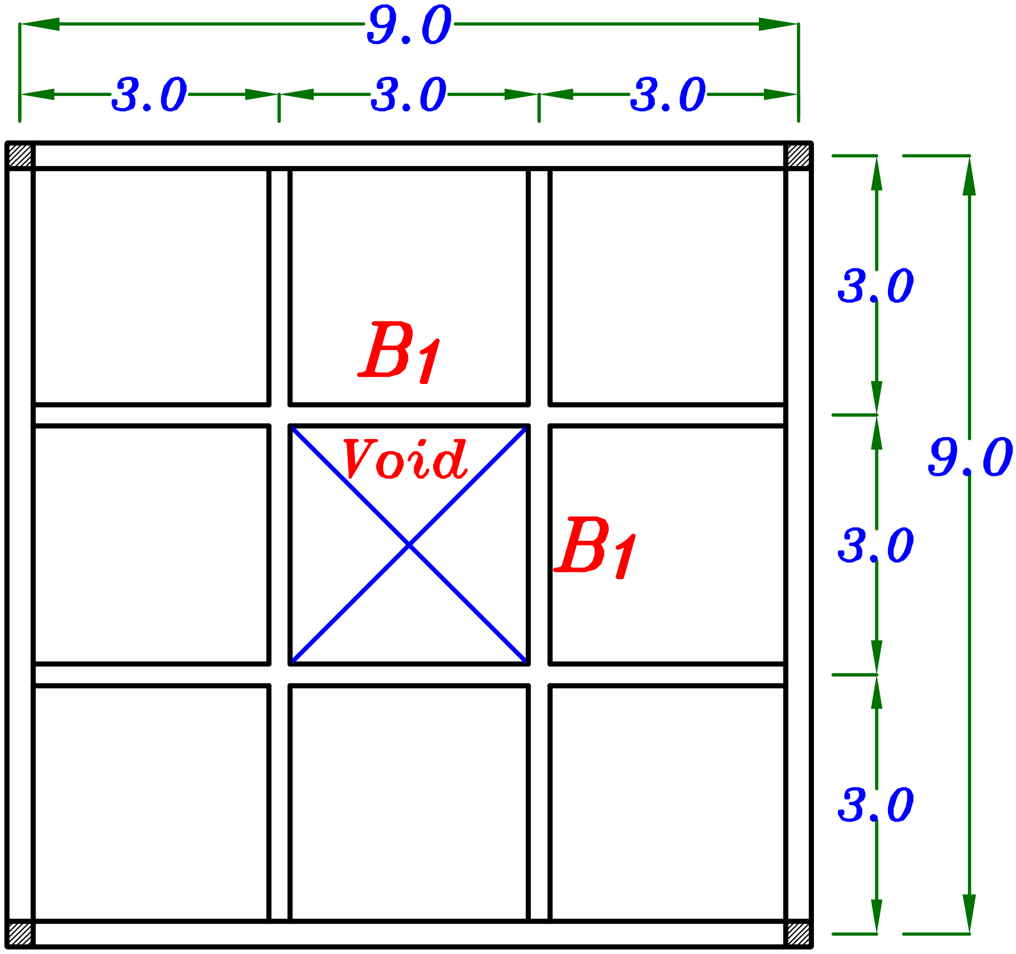
Example.



Data.

$F_{cu} = 25$ N\mm^2 $F_y = 360$ N\mm^2

F.C. = 1.5 kN\m^2 L.L. = 3.0 kN\m^2

Req.

1. Design The Panelled Beam B_1
2. Draw Details of RFT. of the beams in elevation & cross sections.

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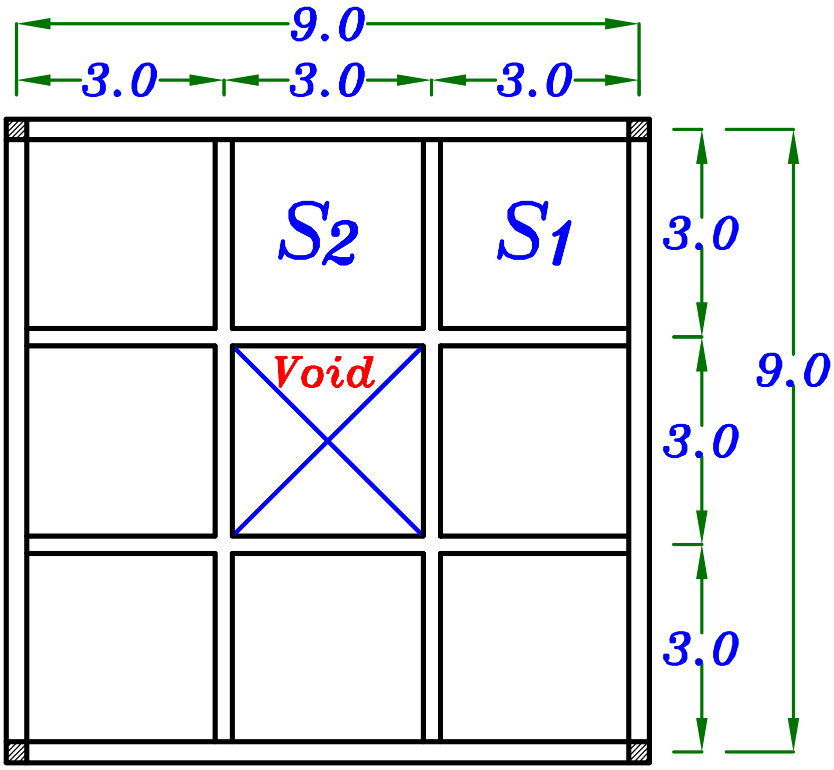
Design of Panelled Beams.



Choose the Thickness of the Slab. (t_s)

S_1 two way $L_s = 3.0\,m$

$$t_s = \frac{3000}{40} = 75 \text{ mm}$$

S_2 two way $L_s = 3.0\,m$

$$t_s = \frac{3000}{35} = 85.7 \text{ mm}$$

Take (t_s) the bigger value $\boxed{t_s = 100\,mm}$

b- Get the Loads on the Slab (w_s).

$$\boxed{w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)} \quad kN\backslash m^2$$

$$w_s = 1.4\,(0.10 * 25 + 1.50) + 1.6\,(3.0) = 10.4 \quad kN\backslash m^2$$

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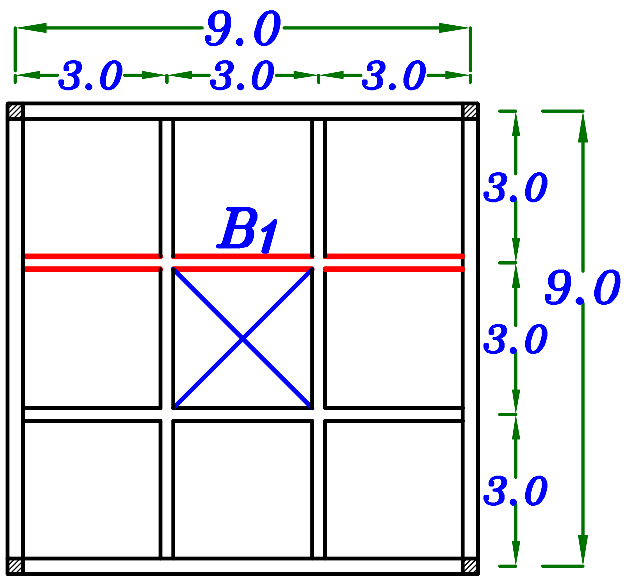
a – Get the Dimensions of the beam. (b,t)

Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{9.0}{16} = 0.56\ m$$

$$= 0.60\ m$$

$$\boxed{b = 0.25\ m} \qquad \boxed{t = 0.60\ m}$$



b – Get the Loads on the Slab. (w_{av})

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s - Void}$$

$$w_{av.} = w_s + \frac{1.4 * b(t - t_s)\,[\text{مجموع أطوال الكمرات الداخلية}] * \gamma_c}{L * L_s - Void}$$

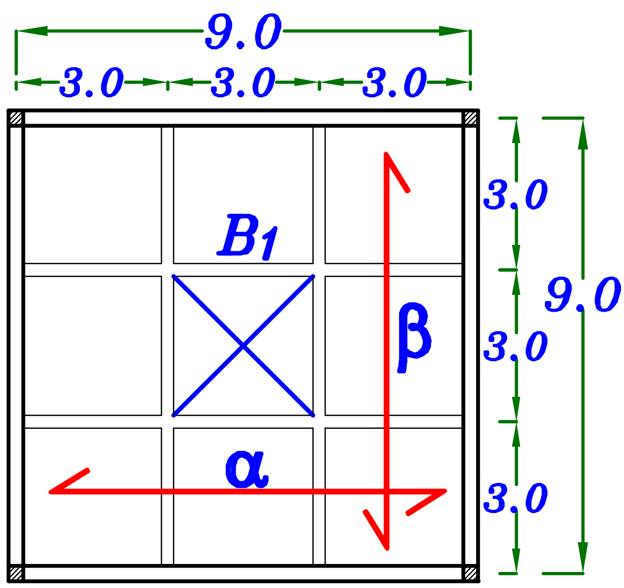
$$w_{av.} = 10.4 + \frac{1.4 * 0.25(0.6 - 0.1)\,[2 * 9.0 + 2 * 9.0] * 25}{9.0 * 9.0 - 3.0 * 3.0} = 12.58\ kN\backslash m^2$$

c – Calculate α, β By using Grashoff.

$$r = \frac{m\ L}{m'\ L_s} = \frac{(1.0)\ 9.0}{(1.0)\ 9.0} = 1.0$$

$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.0)^4}{1 + (1.0)^4} = 0.50$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.0)^4} = 0.5$$



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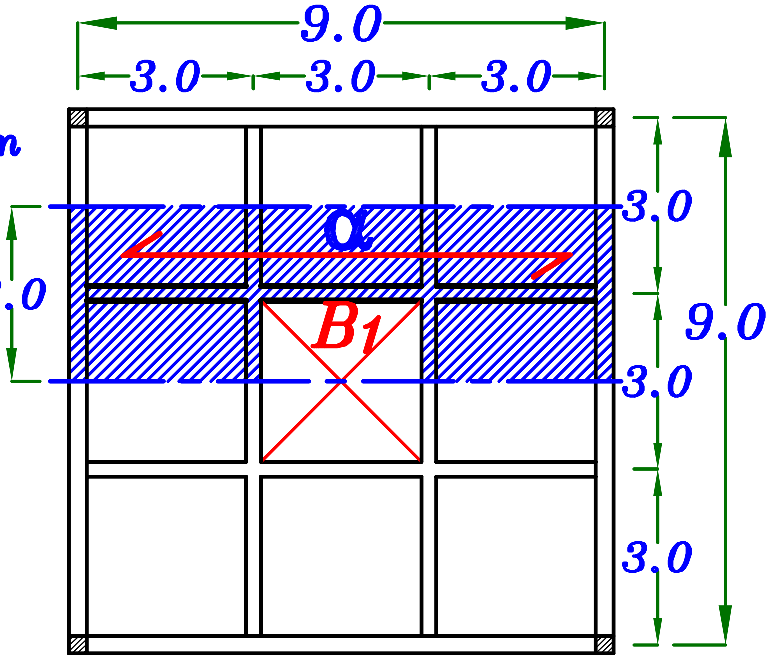
B_1 α Direction.



d- Get the Loads on the Panelled Beam & Calculate the B.M.

$a_1 = 3.0\ m$, $a_2 = 1.5\ m$

$$w_1 = w_{av.} * a_1 * \alpha$$

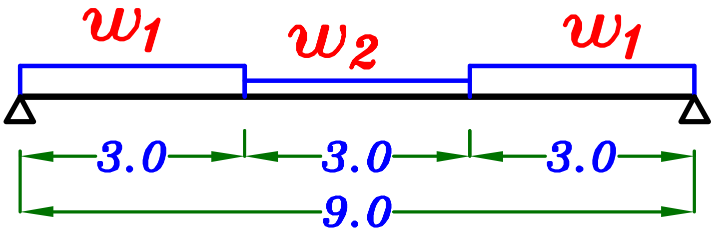
$$= 12.58 * 3.0 * 0.50$$

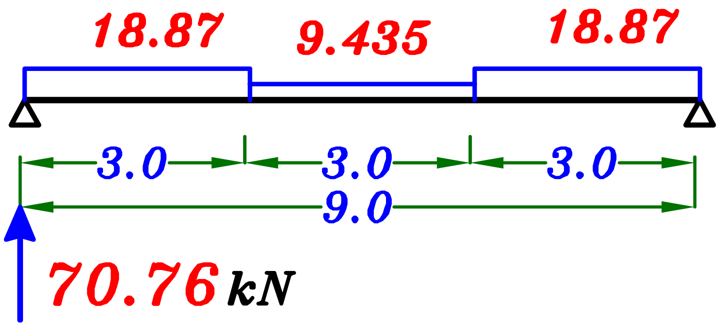
$$= 18.87\ kN\backslash m`$$

$$w_2 = w_{av.} * a_2 * \alpha$$

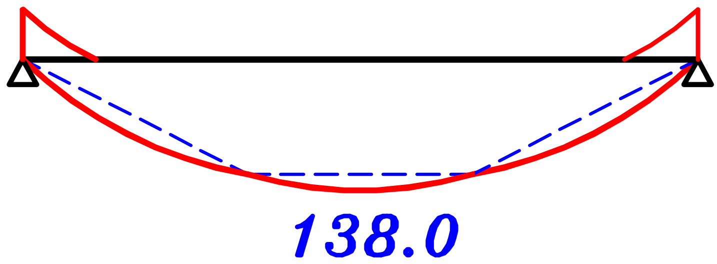
$$= 12.58 * 1.5 * 0.50$$

$$= 9.435\ kN\backslash m`$$



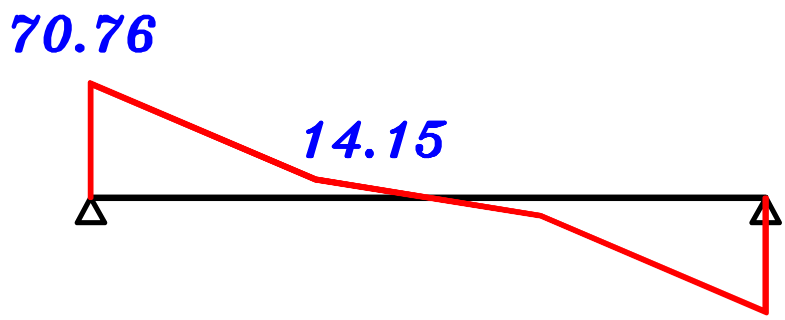


B.M.D.



$$\boxed{M = 138.0\ kN.m}$$

S.F.D.



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e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

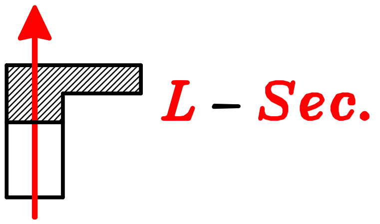
$$X = 3.0\ m\ ,\quad \frac{L}{2} = 4.5\ m$$

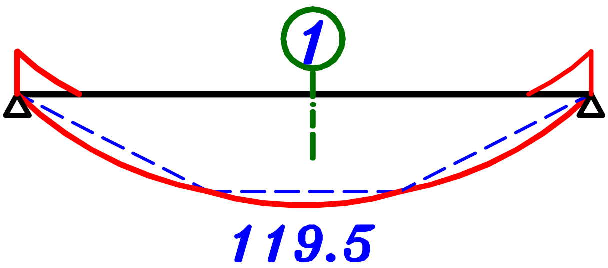
$$\theta_{B_1} = \frac{3.0}{4.5} * 90^\circ = 60^\circ$$

$$M_1 = 138.0 * \frac{\sin 60^\circ}{\sin 90^\circ} = 119.5\ kN.m$$

F– Design the Panelled Beam. B_1

فى الكمرات ال symmetric نصمم كمره واحده فقط و نضع تسليح الكمرتين مثل بعض و يفضل ان نأخذ cover كمره β لانه الاكبر لانه فى التصميم كلما فرضنا ان ال cover اكبر تقل قيمه d فى التصميم فتزيد كميه الحديد .





β Direction. $\therefore$ Cover = 70 mm

$$t = 600\ mm \qquad d = 600 - 70 = 530\ mm$$

$$B = \begin{cases} C.L.-C.L. = 1.5\ m = 1500\ mm \\ 6\,t_s + b = 6*100 + 250 = 850\ mm \\ K\frac{L}{10} + b = 1.0 * \frac{9000}{10} + 250 = 1150\ mm \end{cases}$$

$$\boxed{B = 850\ mm}$$

$$530 = C_1\sqrt{\frac{119.5*10^6}{25*850}} \longrightarrow C_1 = 7.04 \longrightarrow J = 0.826$$

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$$A_s = \frac{119.5 * 10^6}{0.826 * 360 * 530} = 758.2 \ mm^2$$

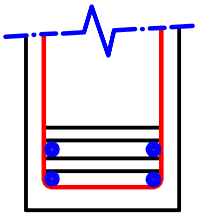
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 758.2 \ mm^2$

$$\mu_{min.} \, b \, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b \, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 530 = 414.06 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.} b \, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 758.2 \ mm^2$ $\quad 4 \phi 22$

$$n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \ \text{bars}$$

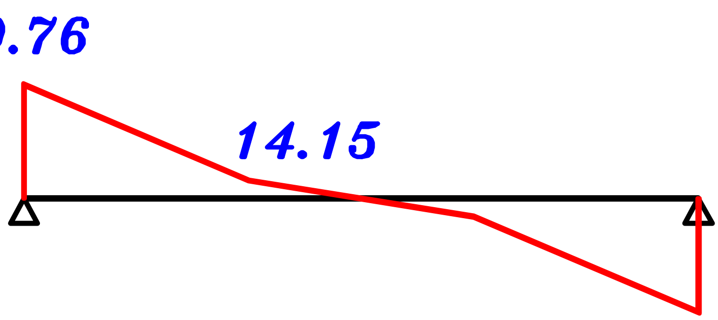
$4 \phi 22$



Stirrup Hangers $= (0.1 \rightarrow 0.2) A_s = (75 \rightarrow 150 \ mm^2)$ $\quad 2 \phi 10$

Check Shear.

S.F.D.



$$q_{cu} = (0.24) \sqrt{\frac{25}{1.50}} = 0.98 \ N \backslash mm^2$$

$$q_{u_{max}} = (0.70) \sqrt{\frac{25}{1.50}} = 2.85 \ N \backslash mm^2$$

$$q_s = \frac{Q_{max}}{b \, d} = \frac{70.76 * 10^3}{250 * 530} = 0.53 \ N \backslash mm^2$$

$\therefore q_s < q_{cu} \qquad \therefore$ Use min. Shear RFT. $\quad 5 \phi 8 \backslash m'$

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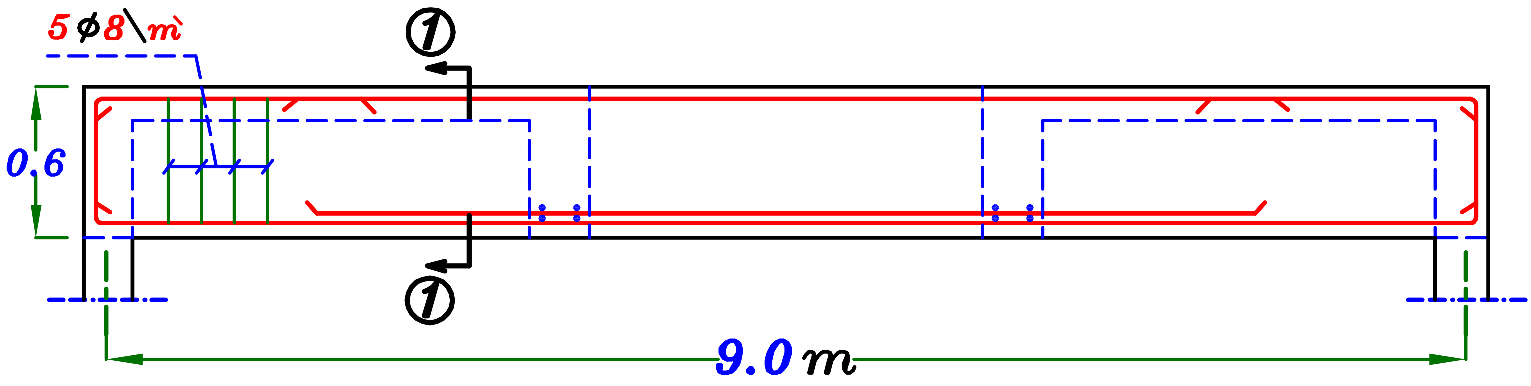
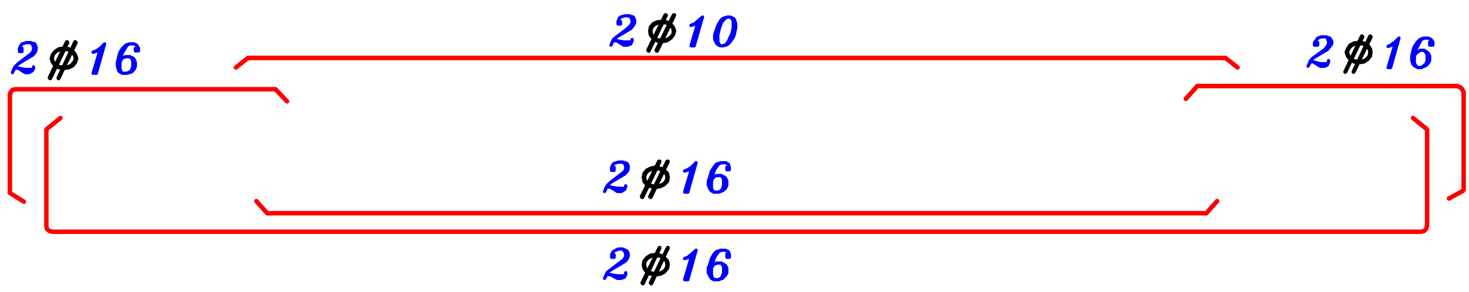
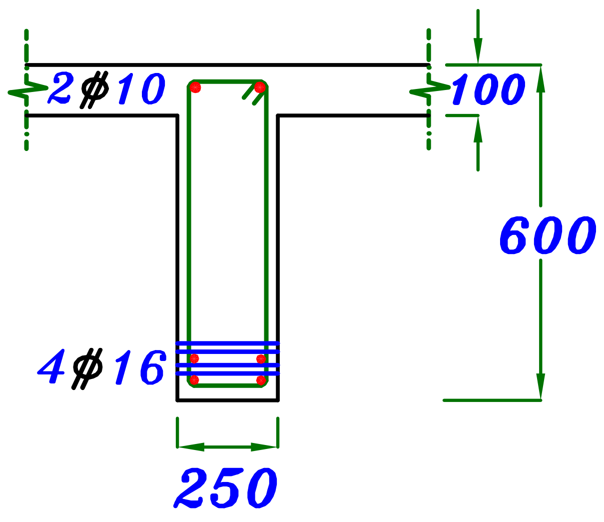
9- Draw Details of RFT. For the Beam B_1







Sec. (1-1)

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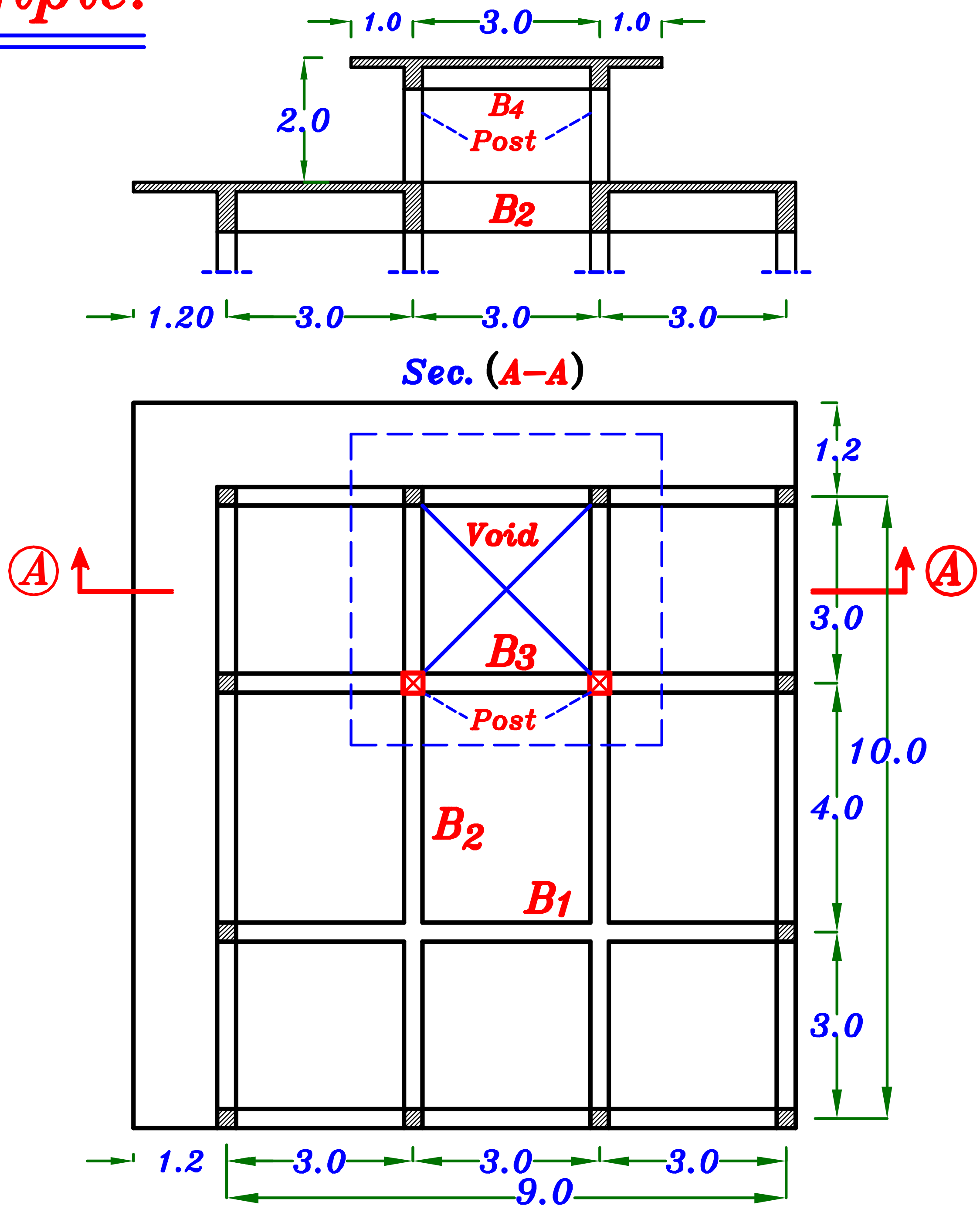
Example.



Data.

$F_{cu} = 25$ N\mm², $F_y = 360$ N\mm²

F.C. = 1.50 kN\m², L.L. = 2.0 kN\m²

Req.

1. Draw a sketch show the RFT. of the slabs. without Calculations.
2. Design the panelled beams B_1, B_2, B_3
3. Draw Details of RFT. of the beams in elevation & cross sections.

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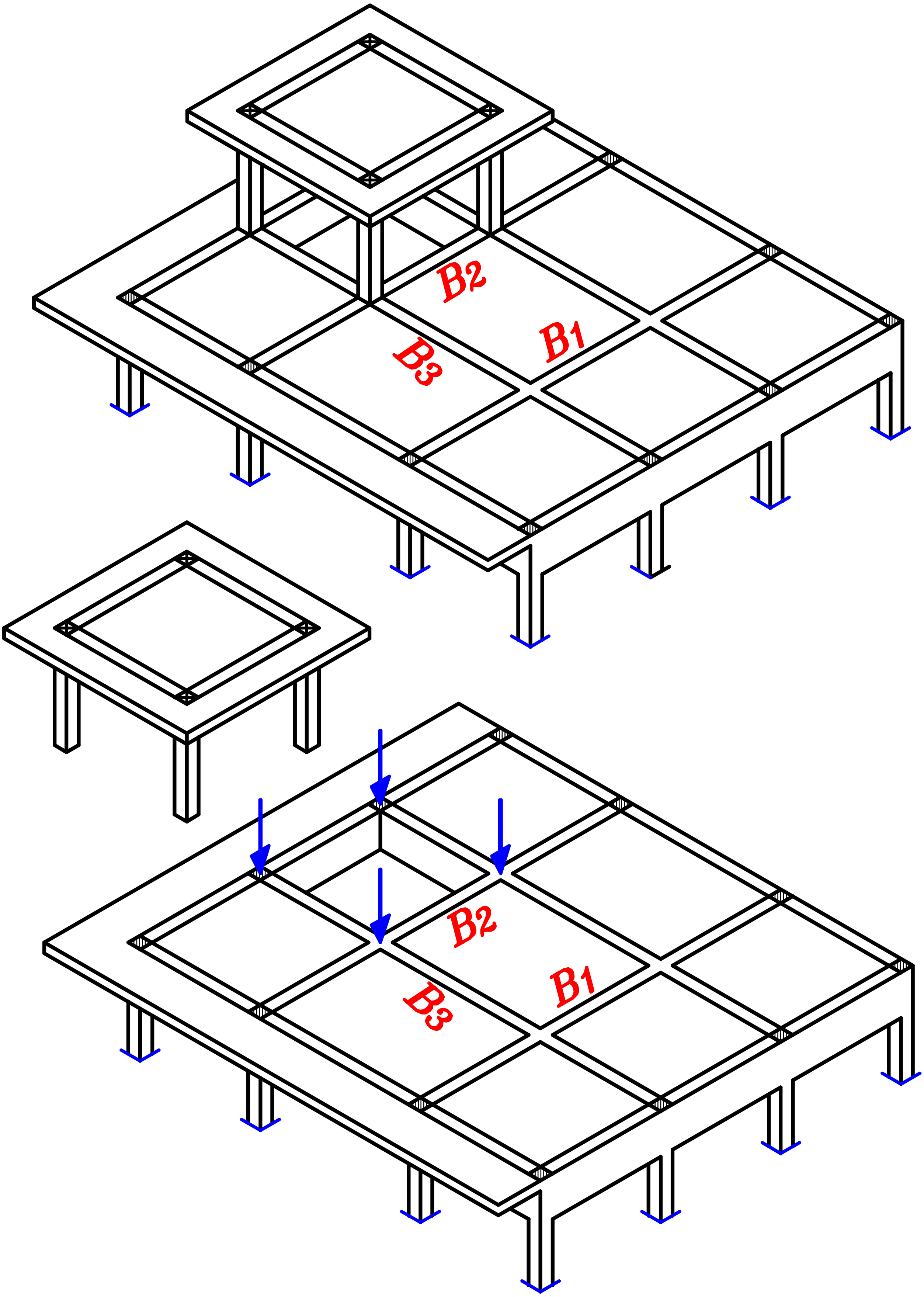

فكره المسأله

الوزن الكلى للشخشيخه يتوزع على **4 Posts** و يكون حمل ال **Post** الواحد حمل مركز
اثنان منهم يحملوا مباشره على الاعمده و الاثنان الاخران يعتبروا حمل مركز محمول على
الكمرات ال B_3 & B_2 **Panelled** و يتوزع الحمل المركز على الكمرات بنسبتى α , β

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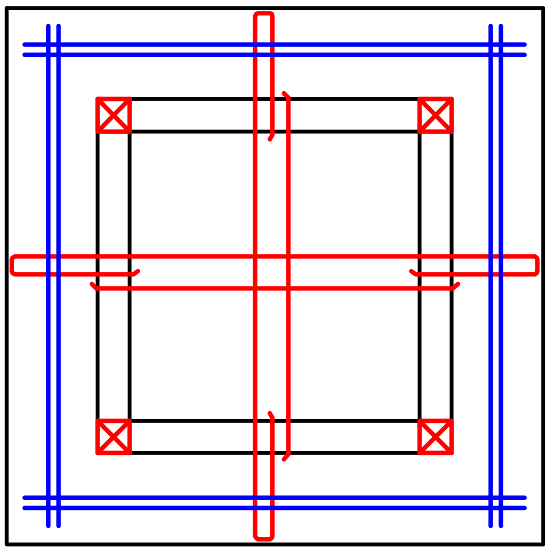
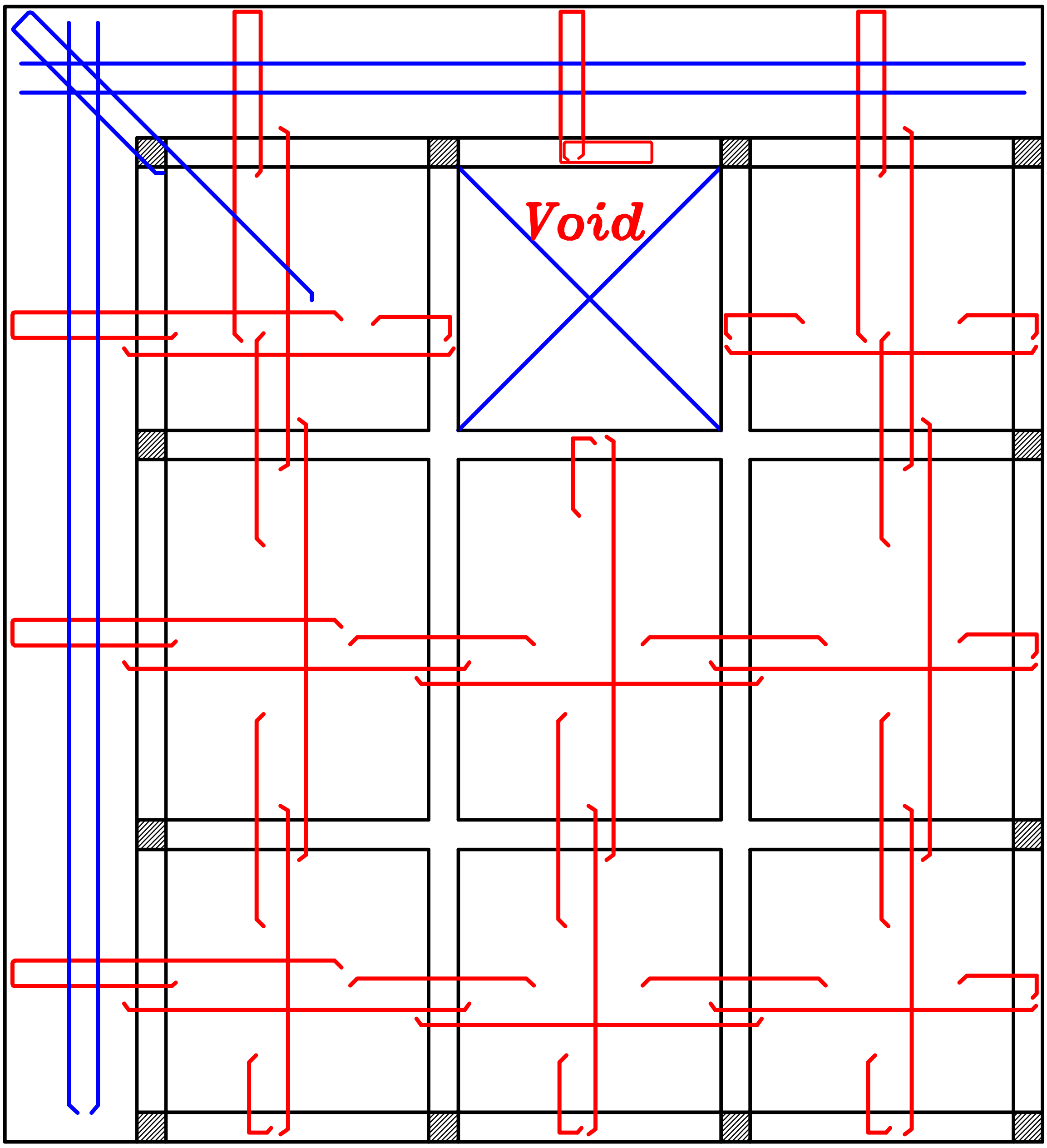
① RFT. of the Slabs.





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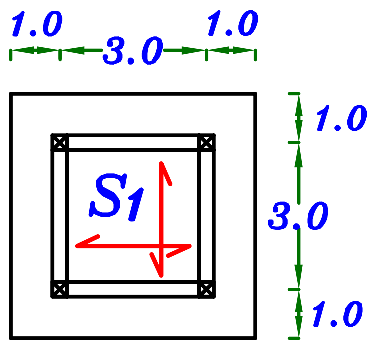
Design of Panelled Beams.



a- Choose the Thickness of the Slab. (t_s) .

S_1 two way $L_s = 3.0\ m$

$$t_s = \frac{3000}{45} = 66.6\ mm$$

S_2 two way $L_s = 3.0\ m$

$$t_s = \frac{3000}{40} = 75.0\ mm$$

S_3 two way $L_s = 3.0\ m$

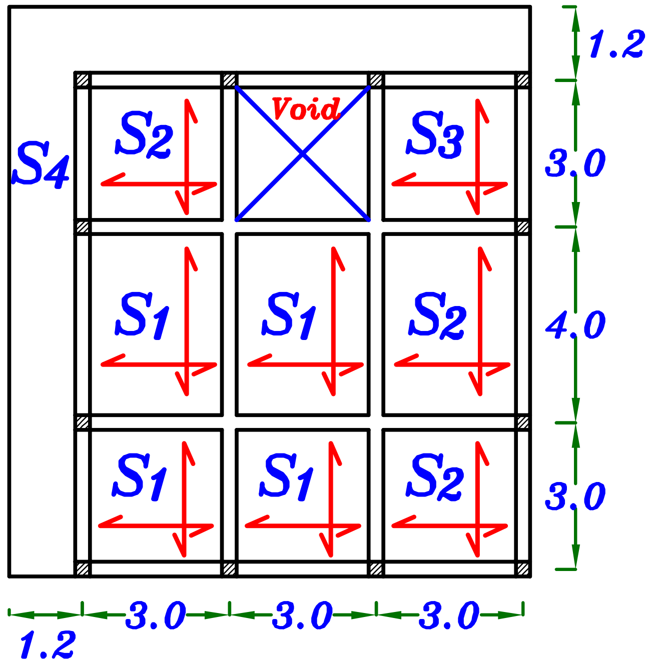
$$t_s = \frac{3000}{35} = 85.7\ mm$$

S_4 Cantilever $L_c = 1.2\ m$

$$t_s = \frac{1200}{10} = 120\ mm$$

Take (t_s) the bigger value

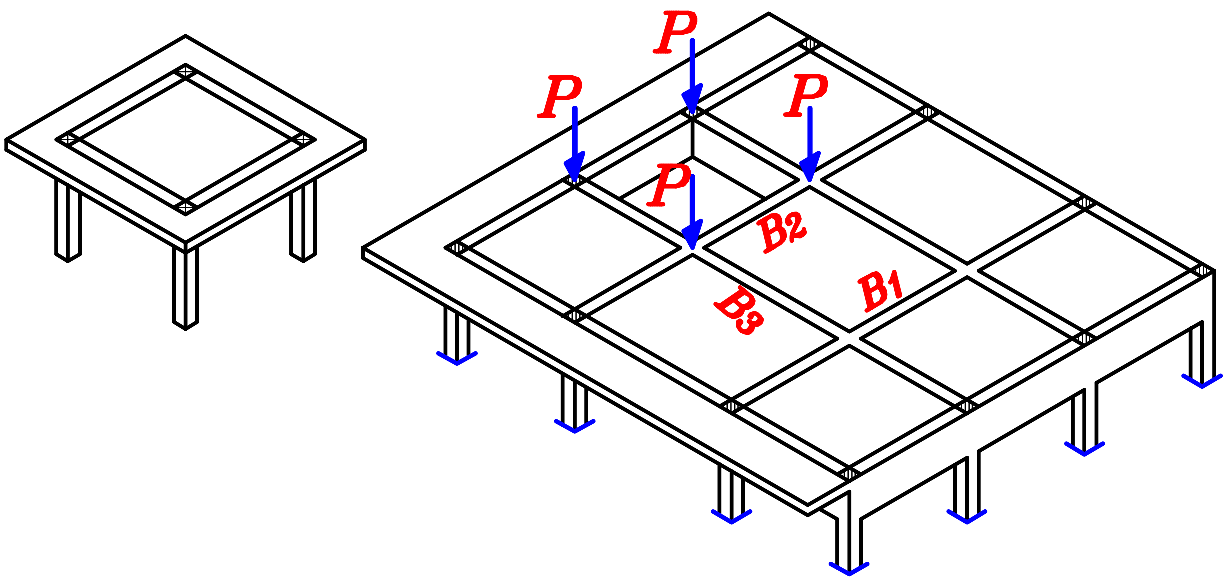
$$\boxed{t_s = 120\ mm}$$



b- Get the Loads on the Slab (w_s).

$$w_s = 1.4\,(0.12 * 25 + 1.50) + 1.6\,(2.0) = 9.50\ kN\backslash m^2$$

الوزن الكلى للشخشيخه يتوزع على 4 Posts و يكون حمل ال Post الواحد حمل مركز
اثنان منهم يحملوا مباشره على الاعمده و الاثنان الاخران يعتبروا حمل مركز محمول على
الكمرات ال Panelled B_2 & B_3 و يتوزع الحمل المركز على الكمرات بنسبتى α , β



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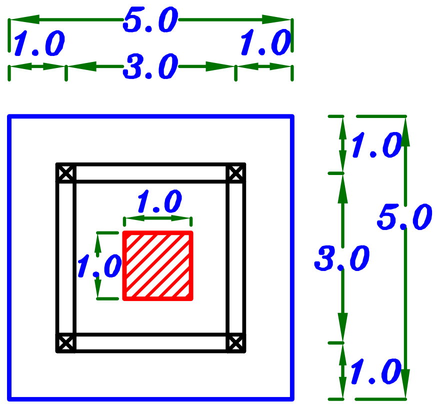
لحساب قيمه P لل Post الواحد نحسب الوزن الكلى للشخشيخه (وزن البلاطه + وزن الكمرات + وزن الPosts)
ثم نقسم الحمل الكلى على ٤ فيكون هذا هو حمل ال Post الواحد

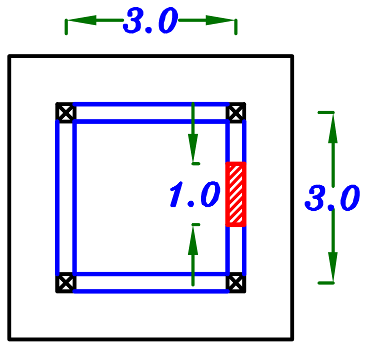


١ – وزن البلاطه = وزن متر مربع × مساحه البلاطه

Total weight of Slab

$W_s * area = 9.50 * (5.0 * 5.0) = 237.5\ kN$

٢ – وزن الكمرات = وزن متر طولى من الكمره × الطول الكلى لل ٤ كمرات

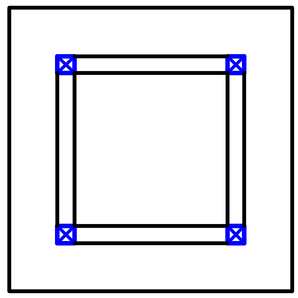


Take $o.w._{(beam)} = 3.0 * 1.4 = 4.20\ kN\backslash m$ (U.L.)

Total weight of Beams

$o.w._{(beam)} * length = 4.20 * (4 * 3.0) = 50.4\ kN$

٣ – وزن ال Posts



Take $o.w._{(Post)} = 3.50\ kN$

Total weight of Posts $= 3.50 * 4 = 14\ kN$

يتوزع الحمل الكلى للشخشيخه على 4 Posts

$$\text{Loads on one Post} = \frac{\sum \text{Weight}}{4.0} = \frac{50.4 + 237.5 + 14}{4.0}$$

$$\boxed{\text{Loads on one Post} = P = 75.5\ kN}$$

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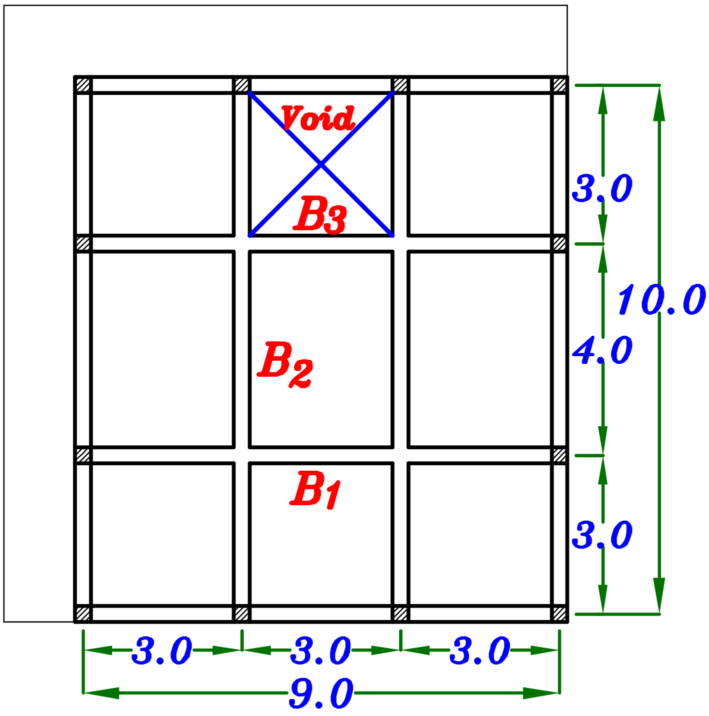
$a-$ Get the Dimensions of the beam. (b, t)



Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{9.0}{16} = 0.56\ m$$

$$= 0.60\ m$$

$$\boxed{b = 0.25\ m} \qquad \boxed{t = 0.60\ m}$$

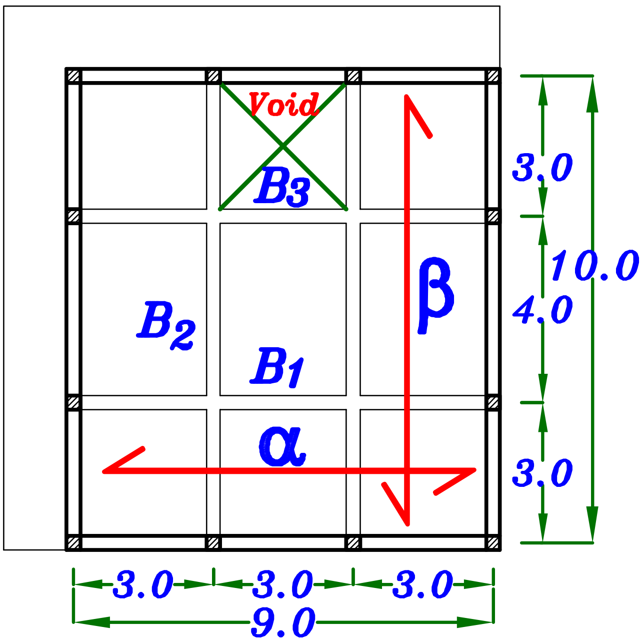
$b-$ Get the Loads on the Slab. (w_{av})

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s - \text{Void}}$$

$$w_{av.} = w_s + \frac{1.4 * b\,(t - t_s)\,[\text{مجموع أطوال الكمرات الداخلية}] * \gamma_c}{L * L_s - \text{Void}}$$

$$w_{av.} = 9.50 + \frac{1.4 * 0.25\,(0.6 - 0.12)\,[2 * 9.0 + 2 * 10.0] * 25}{9.0 * 10.0 - 3.0 * 3.0} = 11.47\ kN\backslash m^2$$

$c-$ Calculate α, β By using Grashoff.



$$r = \frac{m\,L}{m'\,L_s} = \frac{(1.0)\,10.0}{(1.0)\ 9.0} = 1.11$$

$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.11)^4}{1 + (1.11)^4} = 0.603$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.11)^4} = 0.397$$

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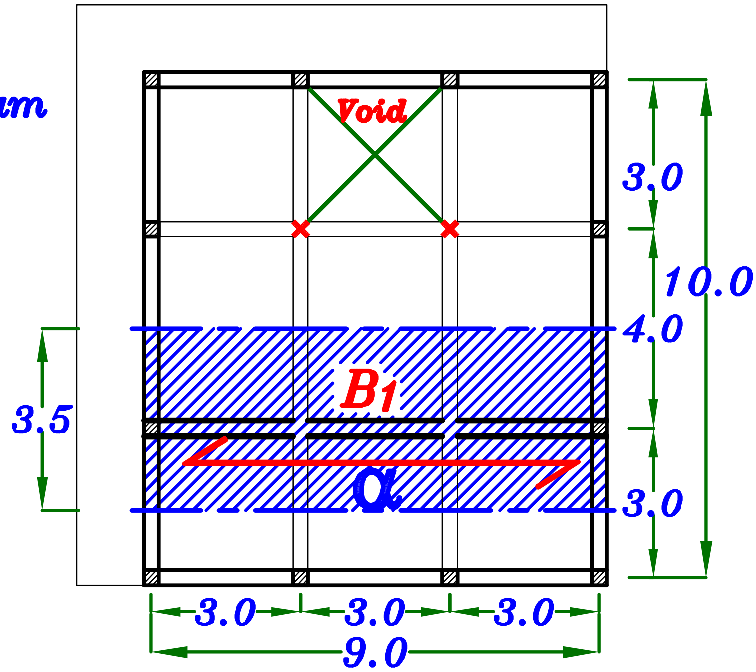
B_1 α Direction.

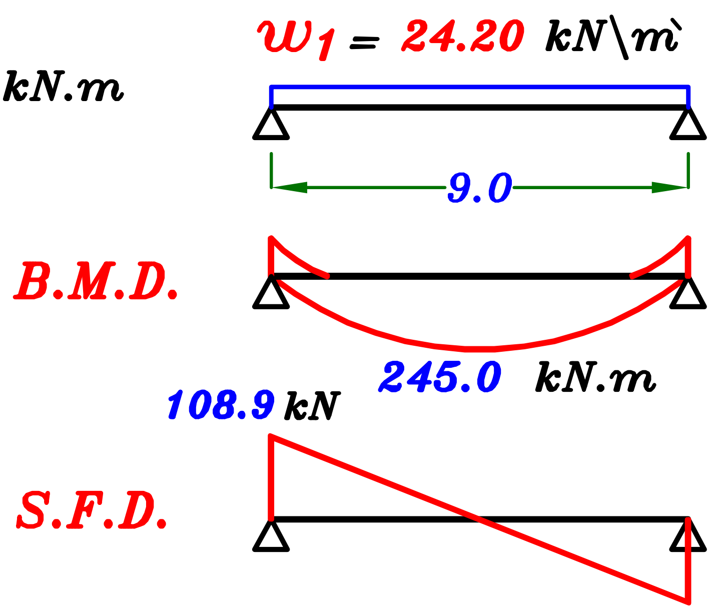
d- Get the Loads on the Panelled Beam & Calculate the B.M.



$$\alpha = 3.5\ m$$

$$w_1 = w_{av.} * \alpha * \alpha$$
$$= 11.47 * 3.5 * 0.603$$
$$= 24.20\ kN\backslash m`$$

$$M = 24.20 * \frac{9.0^2}{8} = 245.0\ kN.m$$



e - Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 3.0\ m\ , \quad \frac{L}{2} = 5.0\ m$$

$$\theta_{B_1} = \frac{3.0}{5.0} * 90^\circ = 54^\circ$$

$$M_1 = 245.0 * \frac{\sin 54^\circ}{\sin 90^\circ} = 198.2\ kN.m$$

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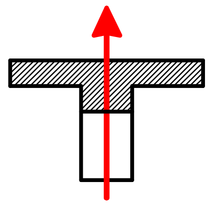
F- Design the Panelled Beam. B_1

α Direction. $\therefore$ Cover = 50 mm



T- Sec.

$t = 600\ mm$ $\qquad d = 60\ - 50 = 550\ mm$

$$B = \begin{cases} C.L.-C.L. = 3.5\ m = 3500\ mm \\ 16\, t_s + b = 16 * 120 + 250 = 2170\ mm \\ K\frac{L}{5} + b = 1.0 * \frac{9000}{5} + 250 = 2050\ mm \end{cases} \qquad \boxed{B = 2050\ mm}$$

$$550 = C_1\sqrt{\frac{198.2 * 10^6}{25 * 2050}} \longrightarrow C_1 = 8.91 \longrightarrow J = 0.826$$

$$A_s = \frac{198.2 * 10^6}{0.826 * 360 * 550} = 1211\ mm^2$$

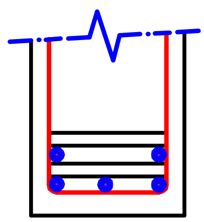
Check $A_{s_{min.}}$

$$A_{s_{req.}} = 1211\ mm^2$$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d$ $\quad \therefore$ Take $A_s = A_{s_{req.}} = 1211\ mm^2$ $\quad$ $5 \phi 18$

$$\therefore\ n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0\ \text{bars}$$



Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (121 \rightarrow 242\ mm^2)$ $\quad$ $2 \phi 10$

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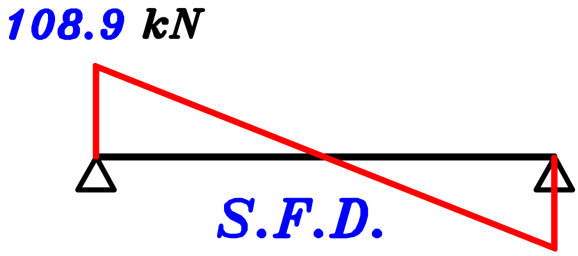
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$



$$q_s = \frac{Q_{max}}{b\,d} = \frac{108.9 * 10^3}{250 * 550} = 0.792 \, N\backslash mm^2 \quad \therefore \, q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $5 \phi 8 \backslash m'$

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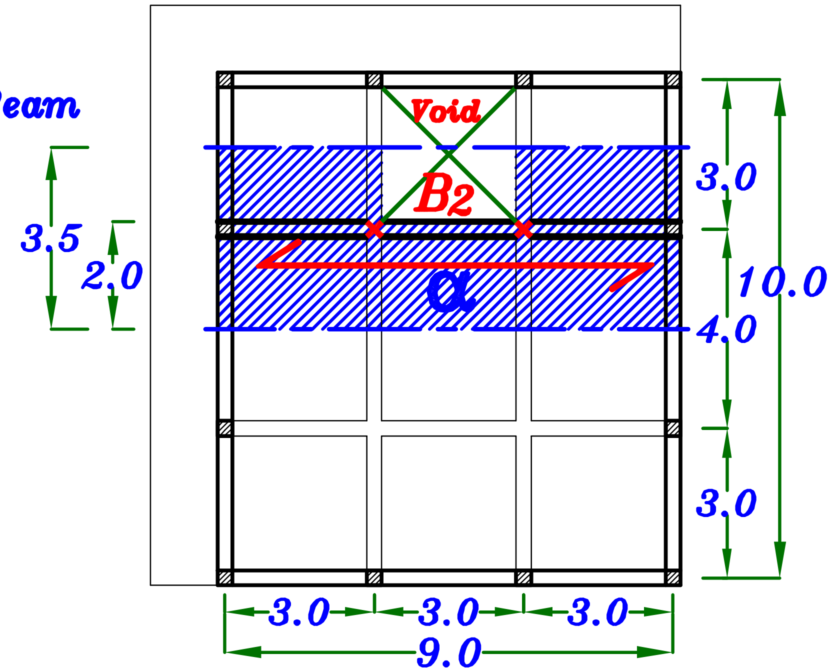
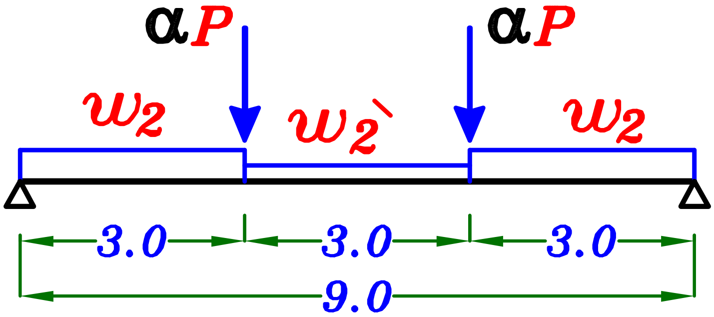
B_2 α Direction.



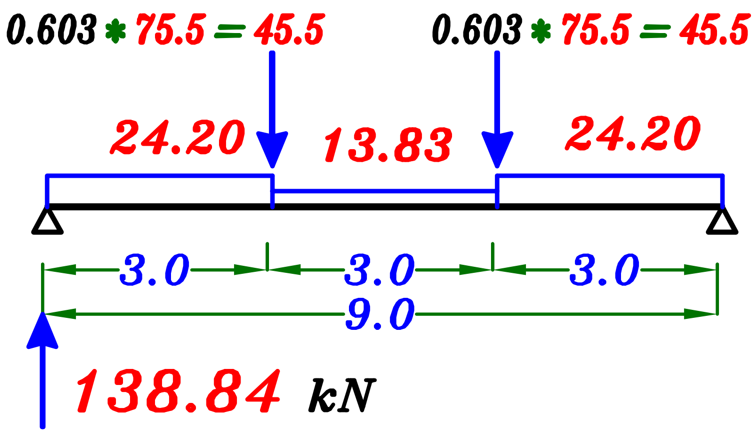
d- Get the Loads on the Panelled Beam & Calculate the B.M.

$a_1 = 3.5\ m$, $a_2 = 2.0\ m$

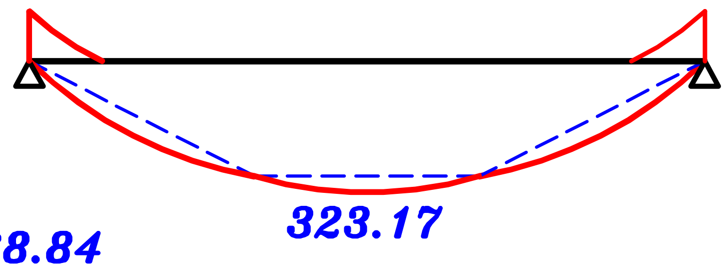
$$w_2 = w_{av.} * a_1 * \alpha$$
$$= 11.47 * 3.5 * 0.603$$
$$= 24.20\ kN\backslash m`$$



$$w_2` = w_{av.} * a_2 * \alpha$$
$$= 11.47 * 2.0 * 0.603$$
$$= 13.83\ kN\backslash m`$$

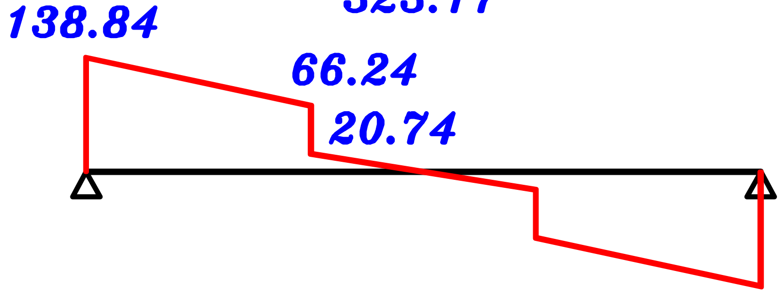


B.M.D.



$$\boxed{M = 323.17\ kN.m}$$

S.F.D.



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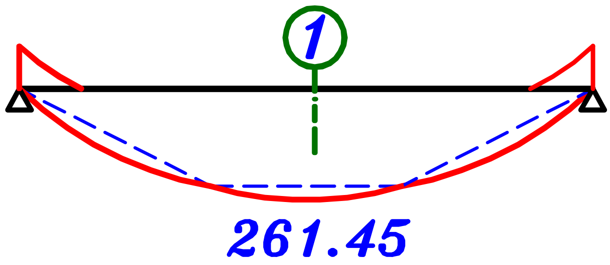
e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin \theta}{\sin 90}\right)$

$$X = 3.0\ m \quad , \quad \frac{L}{2} = 5.0\ m$$

$$\theta_{B_2} = \frac{3.0}{5.0} * 90^\circ = 54^\circ$$

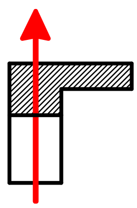
$$M_2 = 323.17 * \frac{\sin 54^\circ}{\sin 90^\circ} = 261.45\ kN.m$$

F– Design the Panelled Beam. B_2



α Direction. $\therefore$ Cover = 50 mm

$t = 600\ mm \qquad d = 600 - 50 = 550\ mm$



L – Sec.

$$B = \begin{cases} C.L.-C.L. = 2.0\ m = 2000\ mm \\ 6\,t_s + b = 6 * 120 + 250 = 970\ mm \\ K\frac{L}{10} + b = 1.0 * \frac{9000}{10} + 250 = 1150\ mm \end{cases} \qquad \boxed{B = 970\ mm}$$

$$550 = C_1 \sqrt{\frac{261.45 * 10^6}{25 * 970}} \longrightarrow C_1 = 5.29 \longrightarrow J = 0.826$$

$$A_s = \frac{261.45 * 10^6}{0.826 * 360 * 550} = 1598\ mm^2$$

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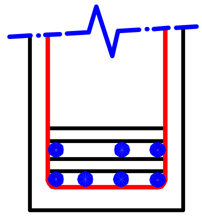
Check $A_{s_{min.}}$

$$A_{s_{req.}} = 1598 \ mm^2$$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7 \ mm^2$$

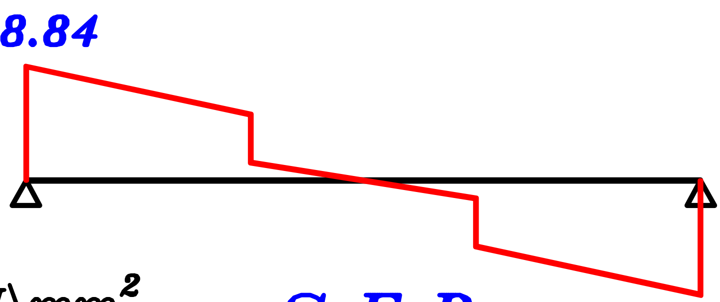
$$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore Take\ A_s = A_{s_{req.}} = 1598 \ mm^2 \qquad 7\phi 18$$

$$n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \ bars$$



$$Stirrup\ Hangers = (0.1 \rightarrow 0.2)\ A_s = (159 \rightarrow 318 \ mm^2) \qquad 3\phi 10$$

Check Shear.



S.F.D.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\, d} = \frac{138.84 * 10^3}{250 * 530} = 1.0 \ N\backslash mm^2 \qquad \therefore q_{cu} < q_s < q_{u_{max}}$$

$$q_s - \frac{q_{cu}}{2} = \frac{n\, A_s (F_y \backslash \gamma_s)}{b\ S} \quad \xrightarrow{Take} \quad n = 2\ ,\ \phi 8 = 50.3 \ mm^2$$

$$1.0 - \frac{0.98}{2} = \frac{2\,(50.3)\,(240 \backslash 1.15)}{250 * S} \longrightarrow S = 164.6 \ mm$$

$$No.\ of\ stirrups\backslash m` = \frac{1000}{S} = \frac{1000}{164.6} = 6.07 \quad Use \quad 7\phi 8 \backslash m`$$

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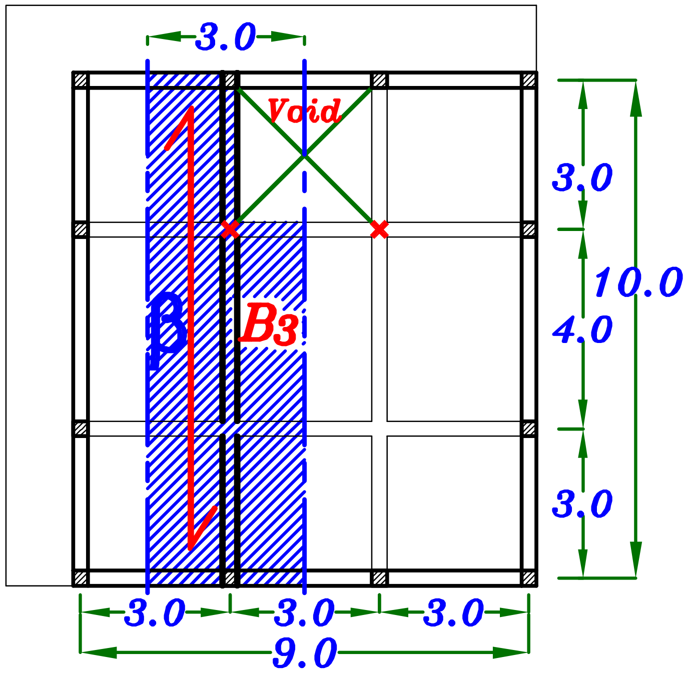
$\underline{\underline{B_3}}$ β Direction.

d– Get the Loads on the Panelled Beam & Calculate the B.M.



$b_1 = 3.0\,m$, $b_2 = 1.5\,m$

$w_3 = w_{av.} * b_1 * \beta$

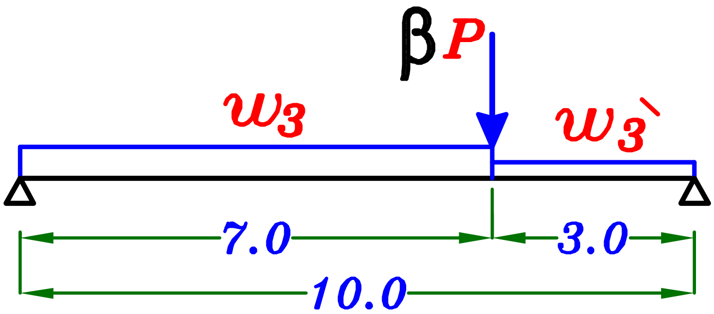
$= 11.47 * 3.0 * 0.397$

$= 13.66$ kN\m\`

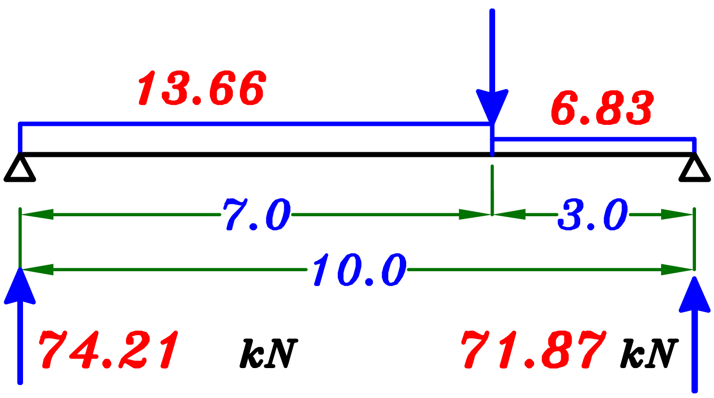
$w_3\text{\`} = w_{av.} * b_2 * \beta$

$= 11.47 * 1.5 * 0.397$

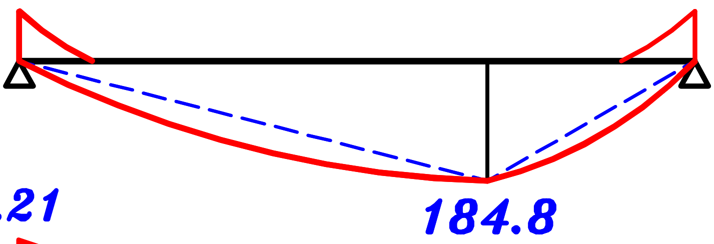
$= 6.83$ kN\m\`



0.397 * 75.5 = 29.97

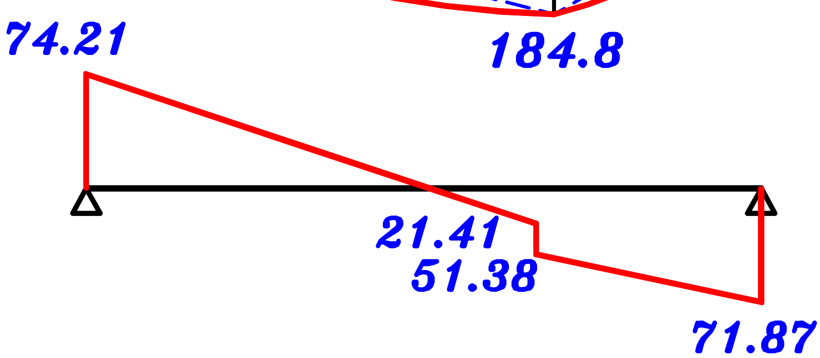


B.M.D.



$\boxed{M = 184.8 \text{ kN.m}}$

S.F.D.



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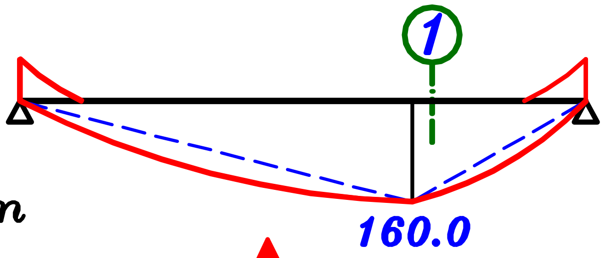
e – Calculate the reduction Factor of the B.M. $\left(\dfrac{\sin\theta}{\sin 90}\right)$

$$X = 3.0\ m \quad , \quad \frac{L}{2} = 4.5\ m$$

$$\theta_{B_2} = \frac{3.0}{4.5} * 90^\circ = 60^\circ$$

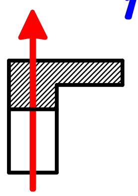
$$M_2 = 184.8 * \frac{\sin 60^\circ}{\sin 90^\circ} = 160.0 \quad kN.m$$

F – Design the Panelled Beam. B_2



β **Direction.** $\therefore$ Cover = 70 mm

$t = 600\ mm \qquad d = 600 - 70 = 530\ mm$



$$B = \begin{cases} C.L.-C.L. = 1.5\ m = 1500\ mm \\ 6\ t_s + b = 6 * 120 + 250 = 970\ mm \\ K\dfrac{L}{10} + b = 1.0 * \dfrac{10000}{10} + 250 = 1250\ mm \end{cases}$$

$$\boxed{B = 970\ mm}$$

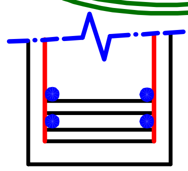
$$530 = C_1 \sqrt{\frac{160.0 * 10^6}{25 * 970}} \longrightarrow C_1 = 6.52 \longrightarrow J = 0.826$$

$$A_s = \frac{160.0 * 10^6}{0.826 * 360 * 530} = 1015\ mm^2$$

Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1015\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 530 = 414.06\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1015\ mm^2$ $\qquad$ **4 ϕ 18**



Stirrup Hangers = $(0.1 \rightarrow 0.2)\ A_s = (101 \rightarrow 202\ mm^2)$

2 ϕ 10

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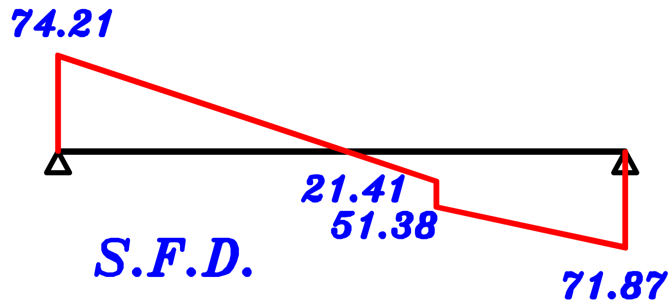
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$



S.F.D.

$$q_s = \frac{Q_{max}}{b\,d} = \frac{74.21 * 10^3}{250 * 530} = 0.56 \ N\backslash mm^2 \qquad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $5\phi8\backslash m'$

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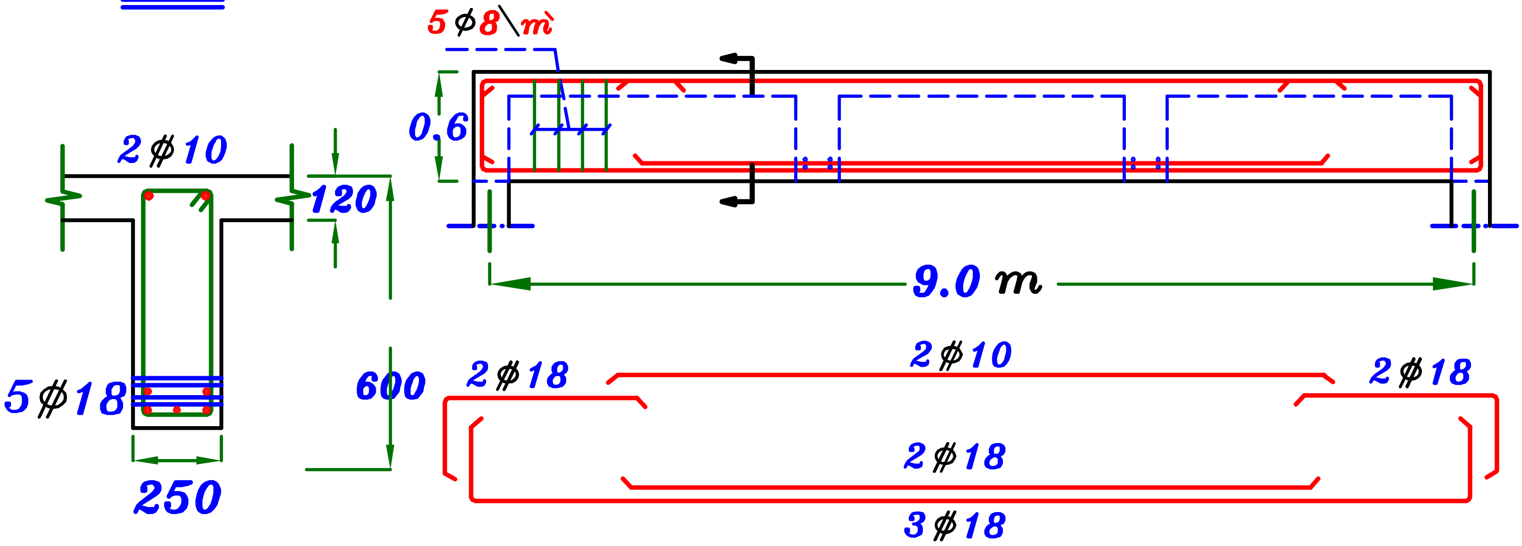
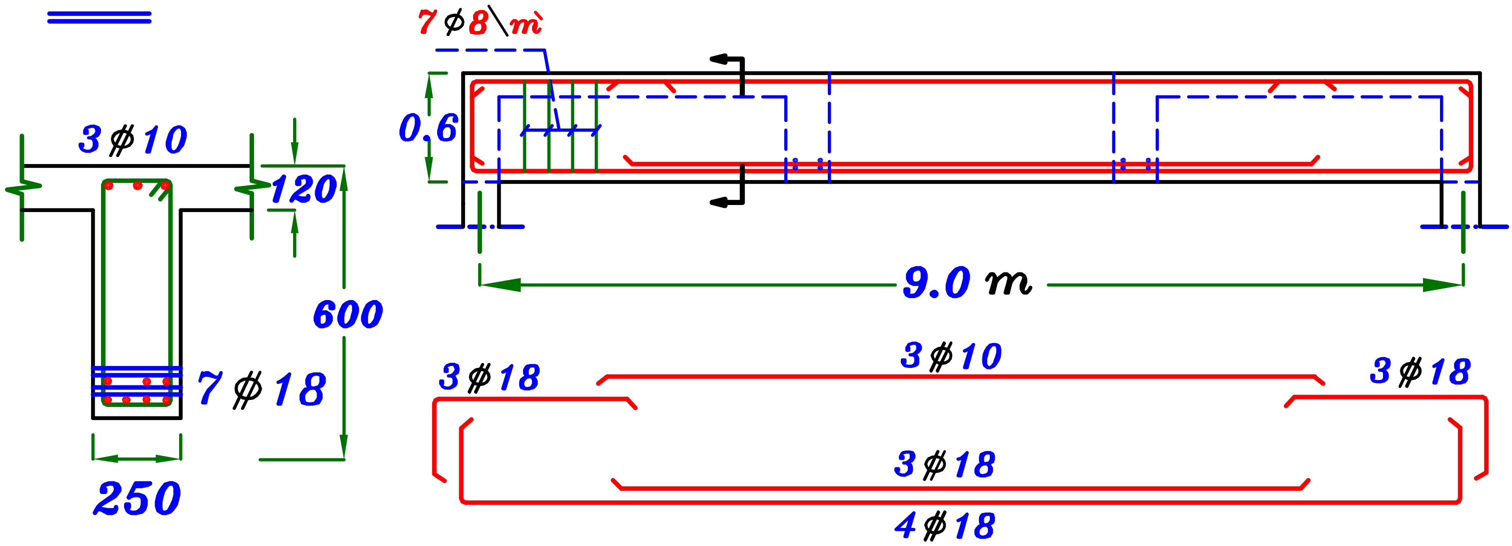
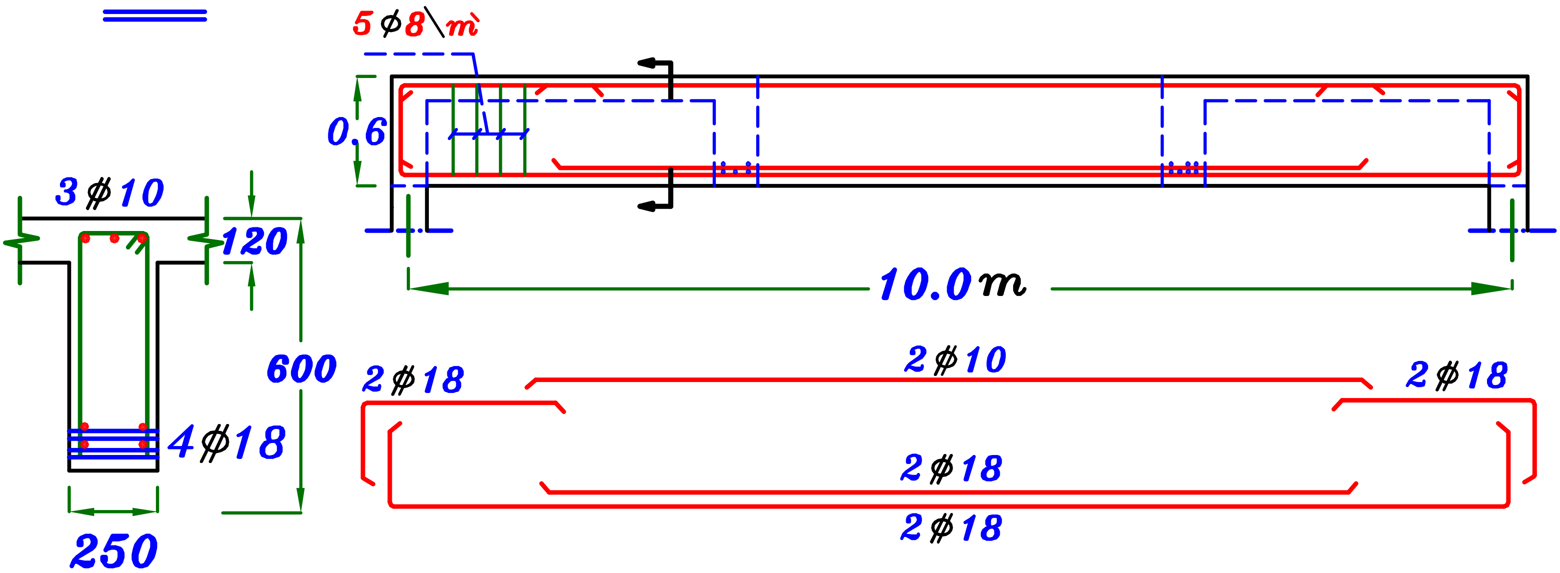
B_1



B_2



B_3



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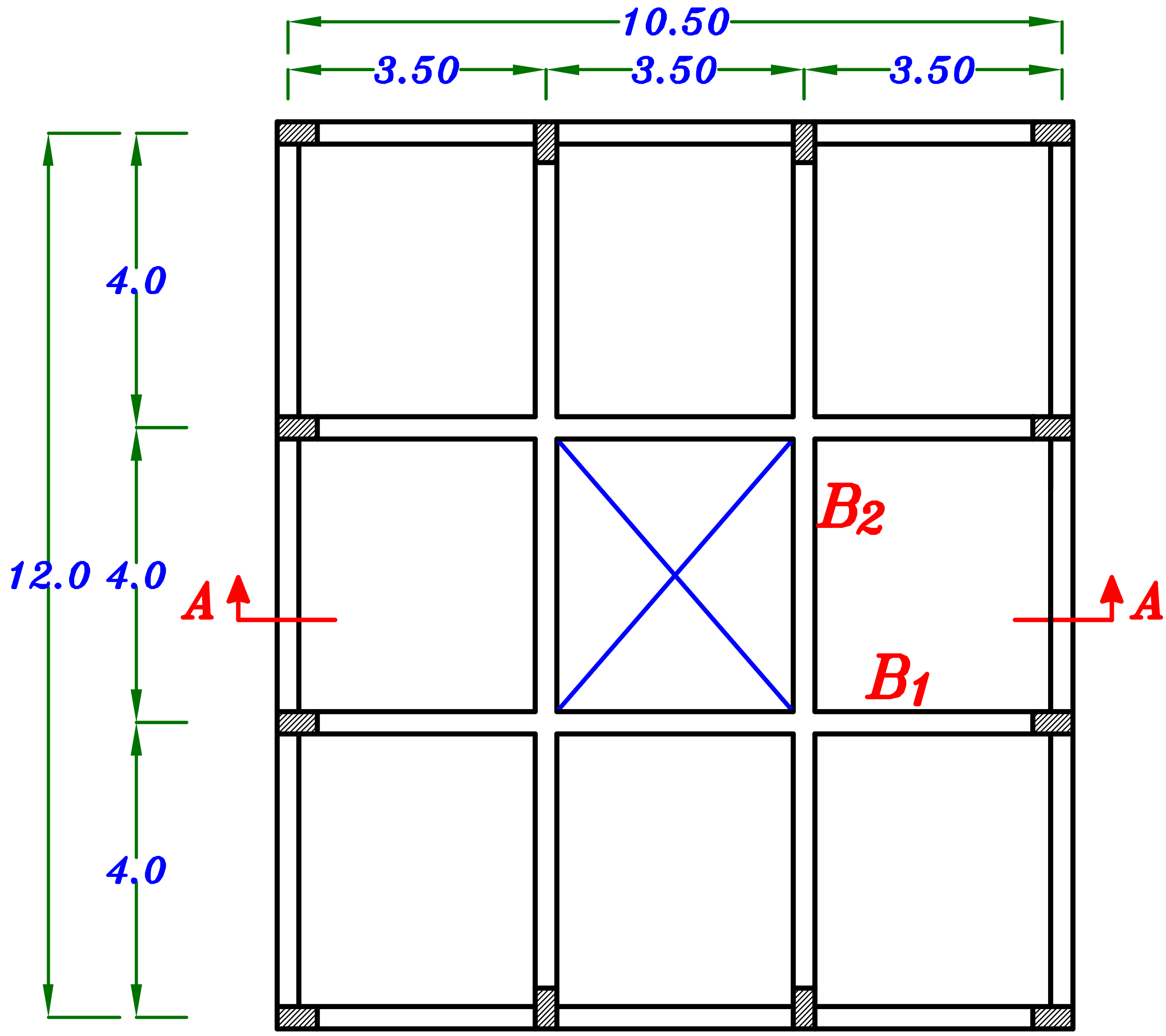
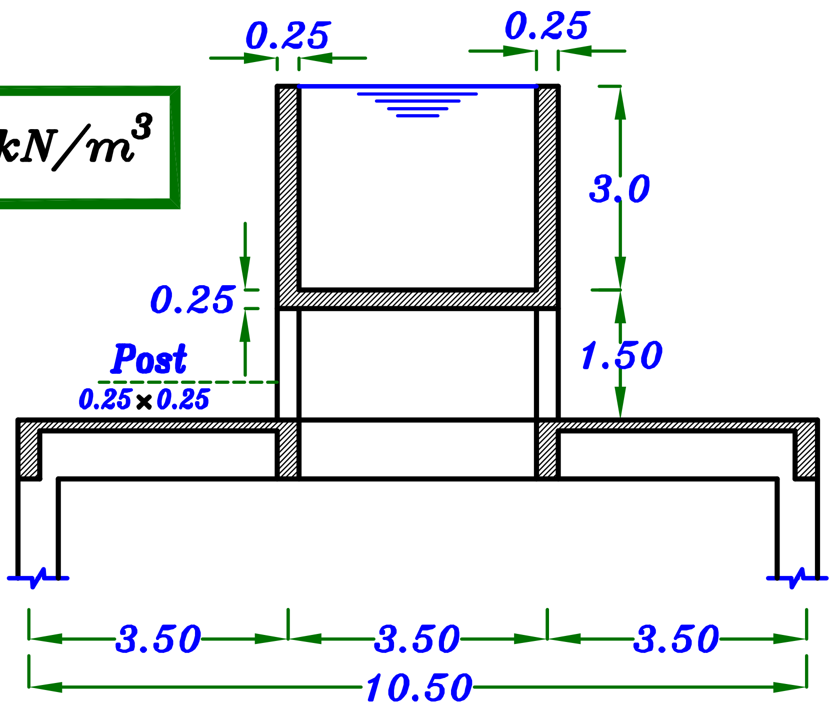
Example.



$\gamma_{water} = 10.0\ kN/m^3$



Design the Panelled Beams B_1, B_2

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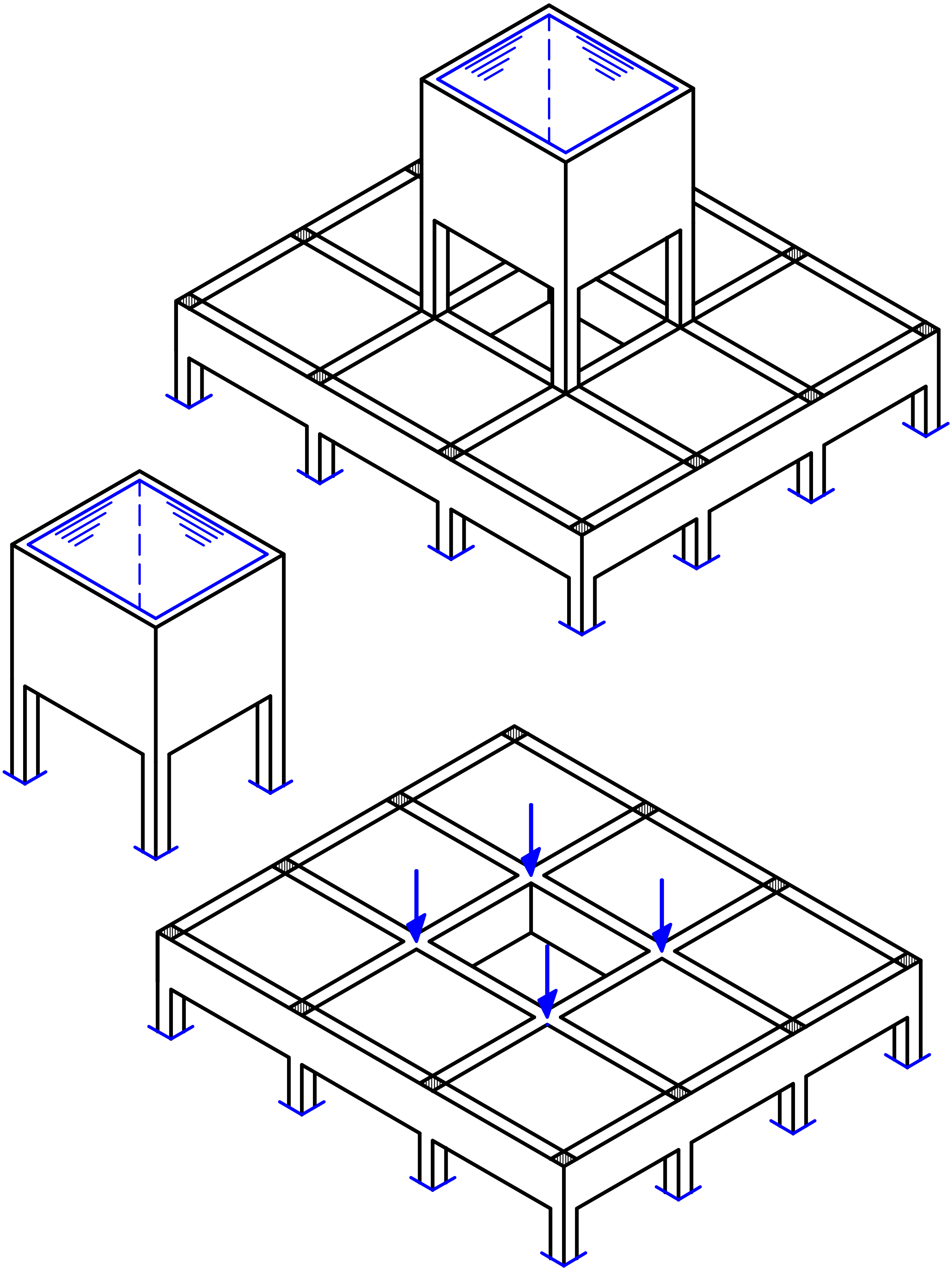
فكره المسألة

الوزن الكلى للخزان الماء و وزن الماء يتوزع على **Posts** **4** بالتساوى

و يكون حمل ال **Post** الواحد حمل مركز محمول على الكمرات ال B_1 & B_2 **Panelled**

و يتوزع الحمل المركز على الكمرات بنسبتى α , β



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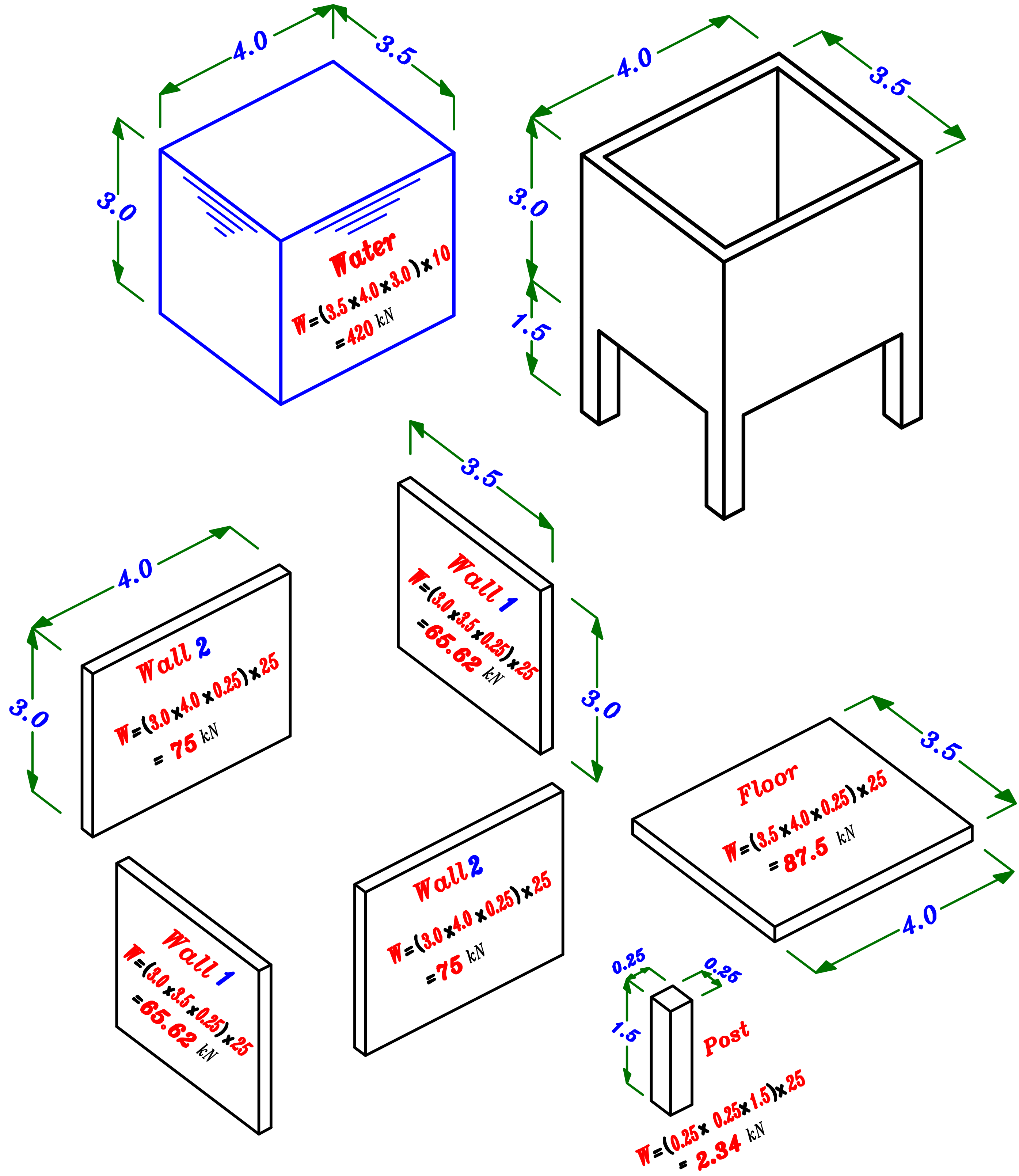

Total weight of the tank

$$= \underbrace{1.5}_{U.L.}\left[\underbrace{420}_{Water} + \underbrace{(2*65.62)}_{Walls\ 1} + \underbrace{(2*75.0)}_{Walls\ 2} + \underbrace{87.5}_{Floor} + \underbrace{(4*2.34)}_{posts}\right] = 1197.15\ kN$$

Load on one post.

$$P = \frac{1197.15}{4.0} = 299.28\ kN$$

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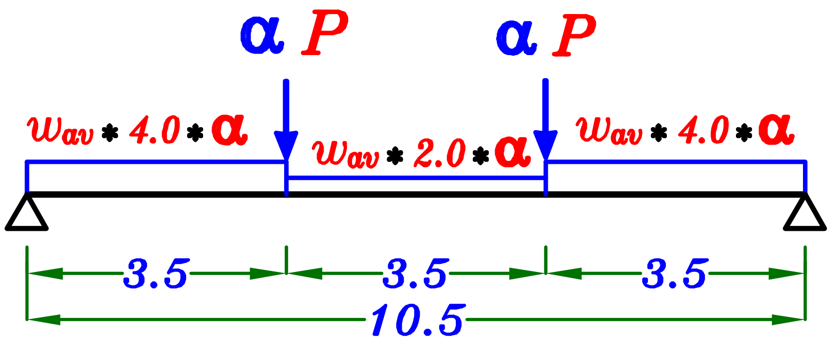
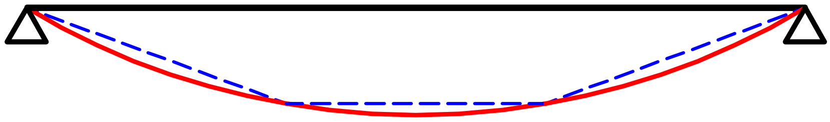
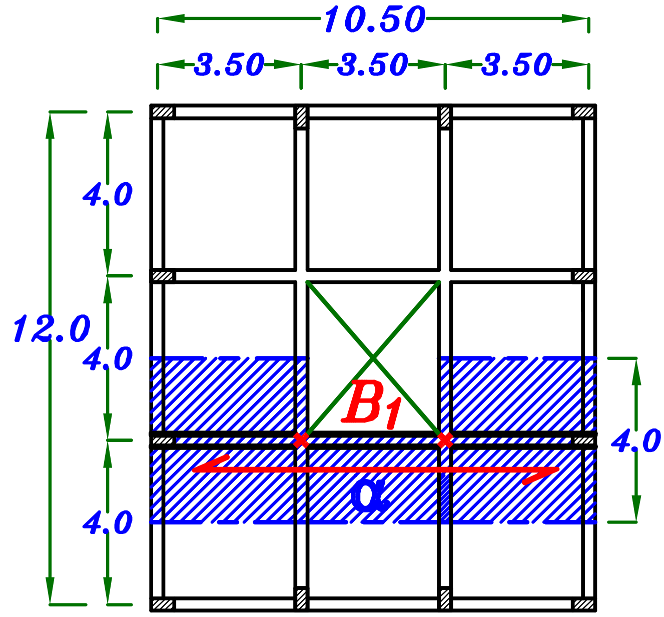
B_1

$$\theta_{B1} = \frac{4.0}{6.0} * 90^\circ = 60^\circ$$

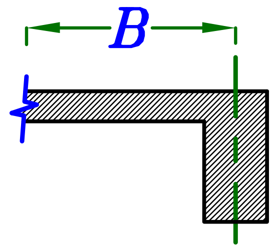




$$M * \frac{\sin 60^\circ}{\sin 90^\circ}$$



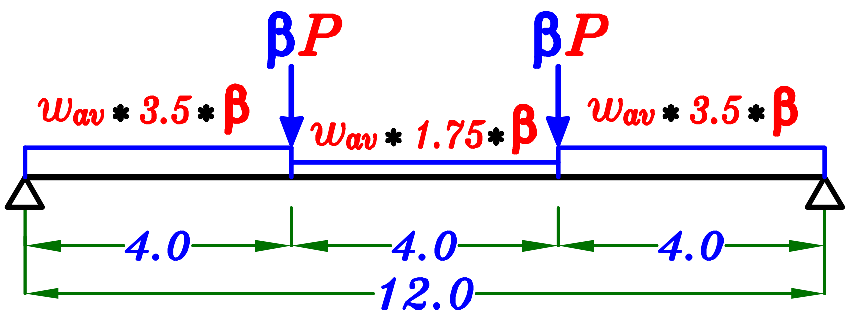
Designed as L-sec.

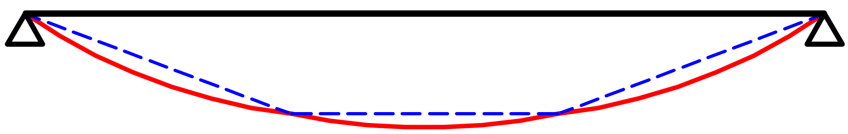


cover = 50 mm

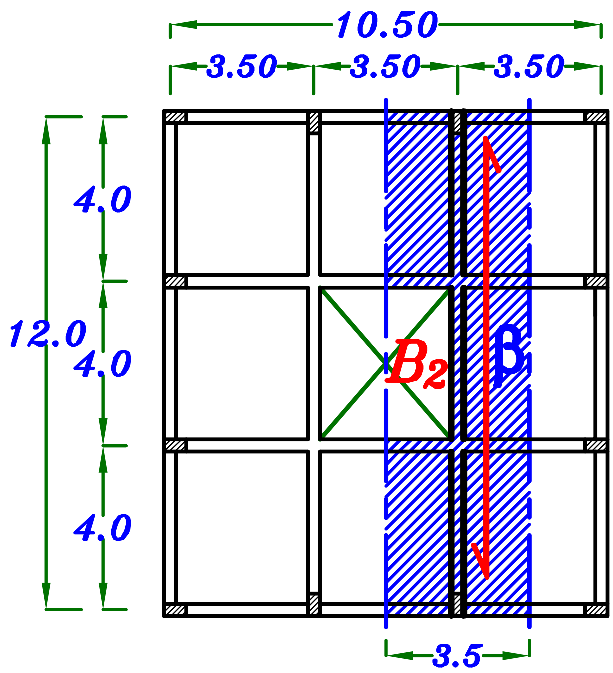
B_2

$$\theta_{B2} = \frac{3.5}{5.25} * 90^\circ = 60^\circ$$

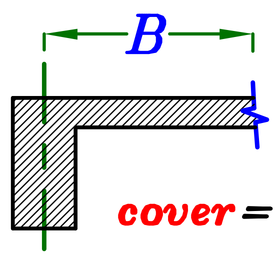




$$M * \frac{\sin 60^\circ}{\sin 90^\circ}$$



Designed as L-sec.



cover = 70 mm

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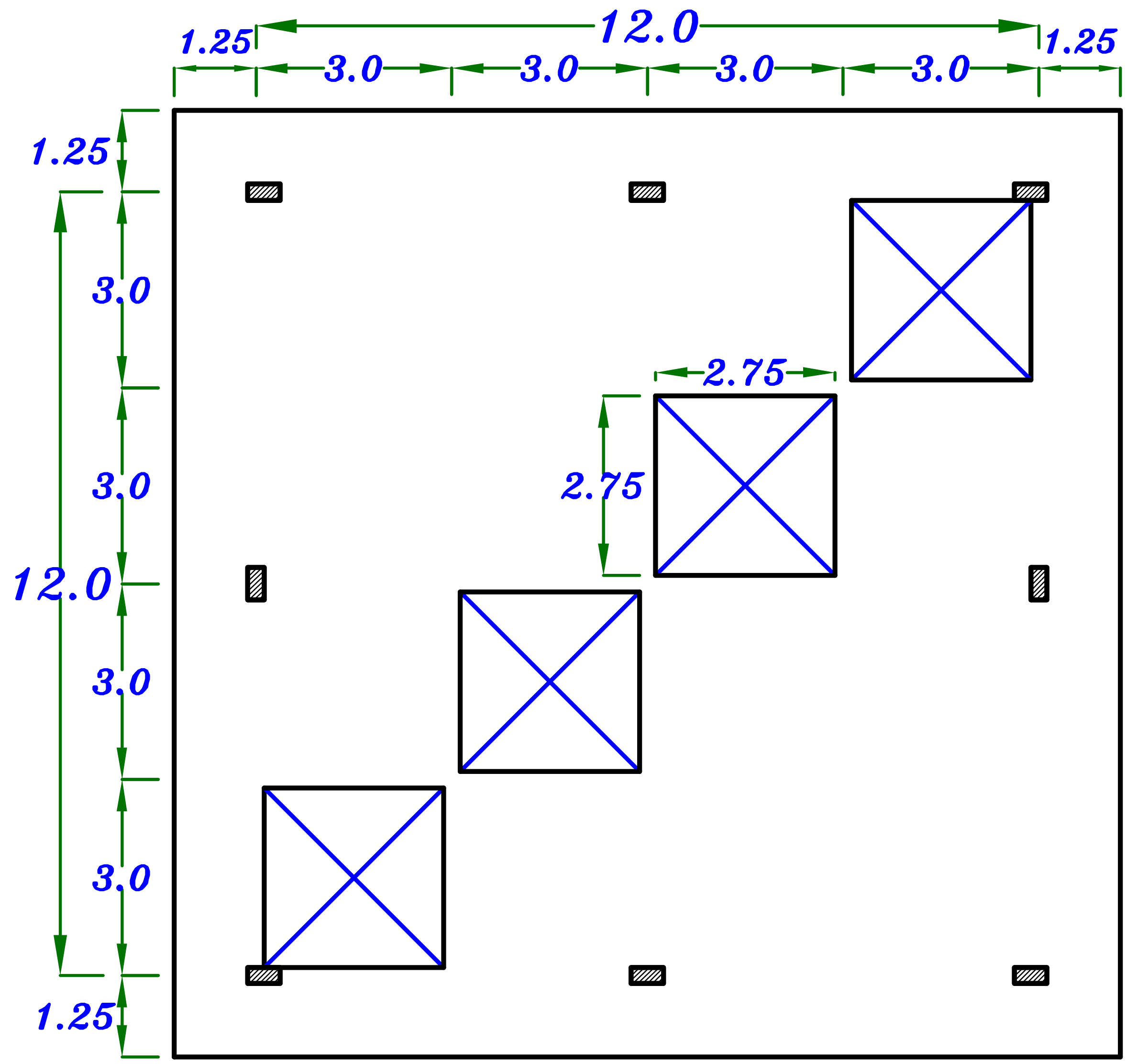
Example.



Data.

$F_{cu} = 25\ N\backslash mm^2$ $\quad$ $F_y = 360\ N\backslash mm^2$

$F.C. + L.L. = 5.0\ kN\backslash m^2$

Req.

1. Choose a system of slabs and panelled beams to cover this area.
2. Design all Panelled Beams.
3. Draw Details of RFT. of the beams in elevation & cross sections.

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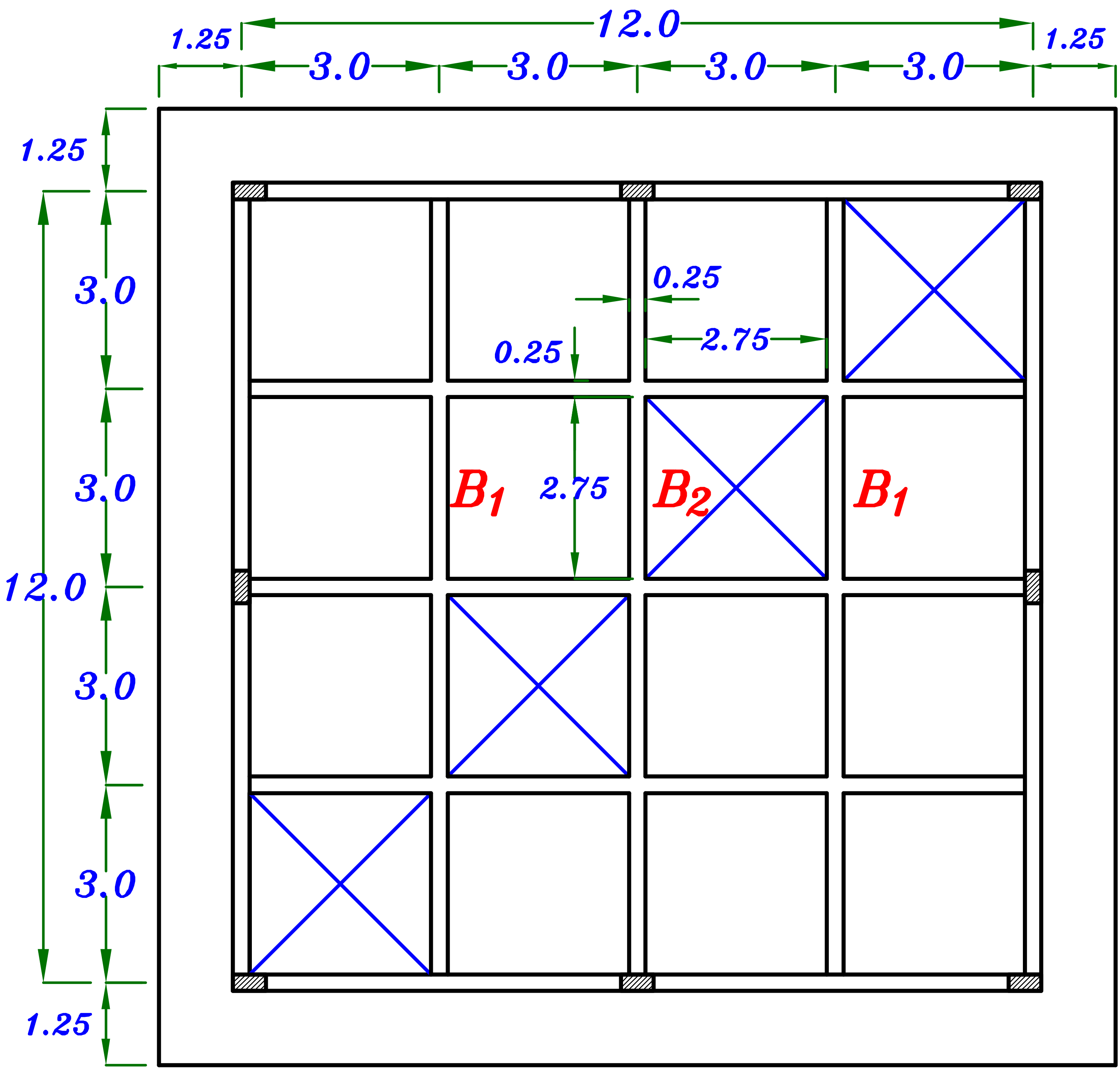
1 *Choose a system of slabs and panelled beams to cover this area.*



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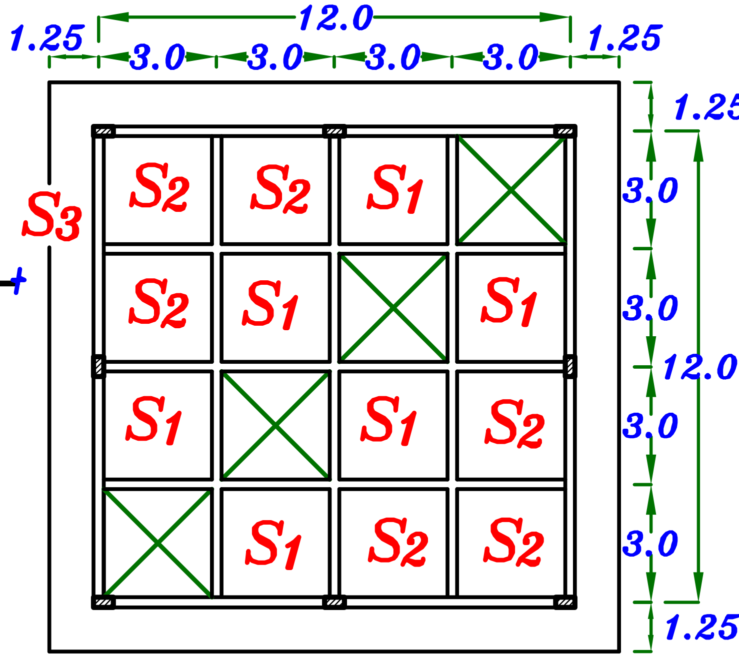
Design of Panelled Beams.

1- Choose the Thickness of the Slab. (t_s).



S_1 two way $L_s = 3.0\ m$

$t_s = \frac{3000}{40} = 75.0\ mm$

S_2 two way $L_s = 3.0\ m$

$t_s = \frac{3000}{45} = 66.6\ mm$

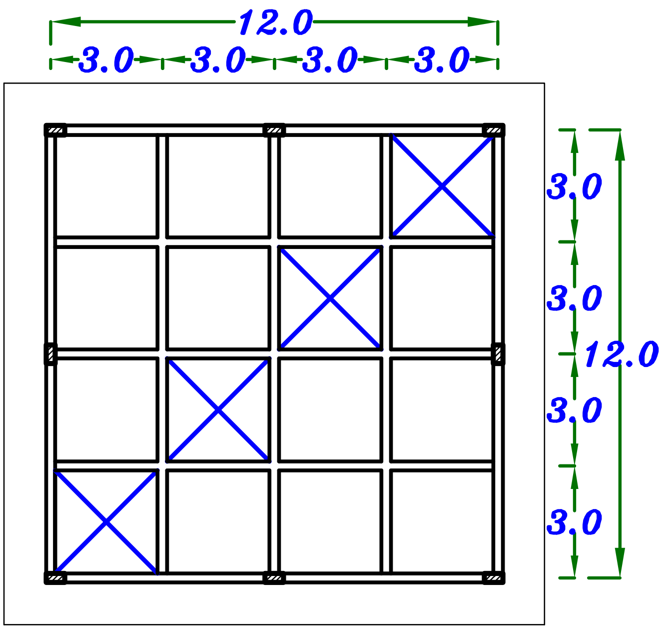
S_3 Cantilever $L_c = 1.25\ m$

$t_s = \frac{1250}{10} = 125\ mm$

Take (t_s) the bigger value $\boxed{t_s = 140\ mm}$

2- Get the Loads on the Slab (w_s).

$w_s = 1.5\,(0.14 * 25 + 5.0) = 12.75\ kN\backslash m^2$



a- Get the Dimensions of the beam. (b, t)

Take $b = 0.25\ m$

$t = \frac{L_s}{16} = \frac{12.0}{16} = 0.75\ m$

$\boxed{b = 0.25\ m}$ $\boxed{t = 0.75\ m}$

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b – Get the Loads on the Slab. (w_{av})

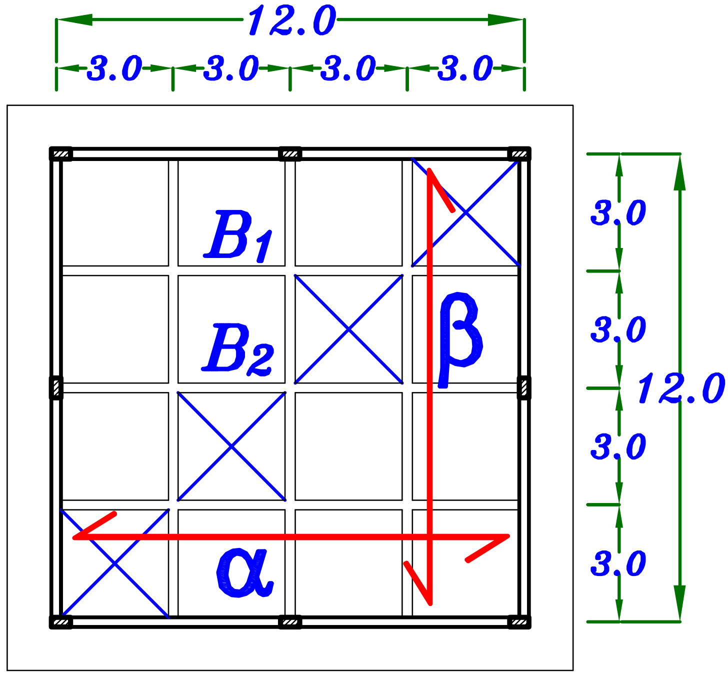
$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s - \text{Voids}}$$

$$w_{av.} = w_s + \frac{1.4 * b\,(t - t_s)\left[\text{مجموع أطوال الكمرات الداخلية}\right] * \gamma_c}{L * L_s - \text{Voids}}$$

$$w_{av.} = 12.75 + \frac{1.4 * 0.25\,(0.75 - 0.14)\,[3 * 12.0 + 3 * 12.0] * 25}{12.0 * 12.0 - 4.0 * (3.0 * 3.0)} = 16.31 \quad kN\backslash m^2$$

c – Calculate α, β By using Grashoff.

$$r = \frac{m\,L}{m'\,L_s} = \frac{(1.0)\,12.0}{(1.0)\,12.0} = 1.0$$



$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.0)^4}{1 + (1.0)^4} = 0.50$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.0)^4} = 0.50$$

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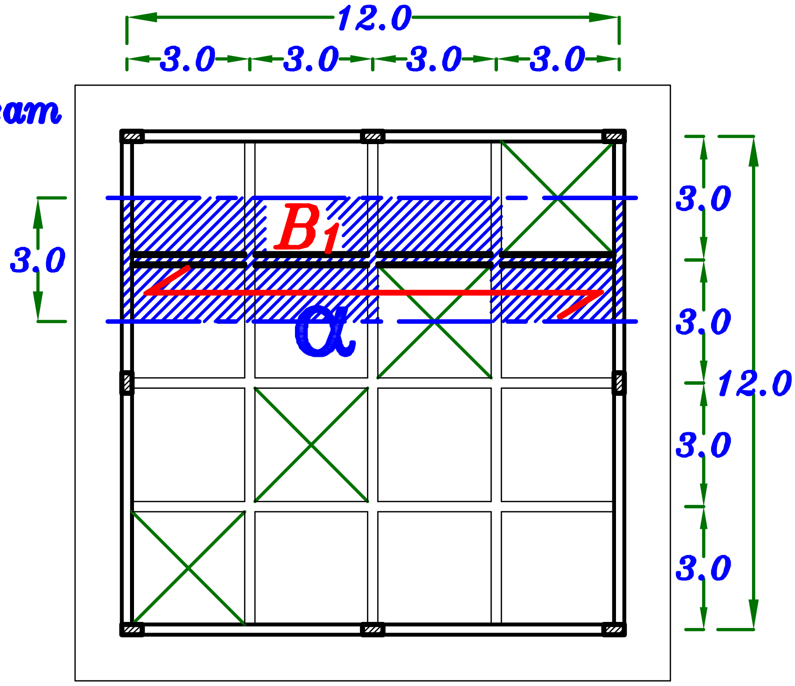
B_1 α Direction.

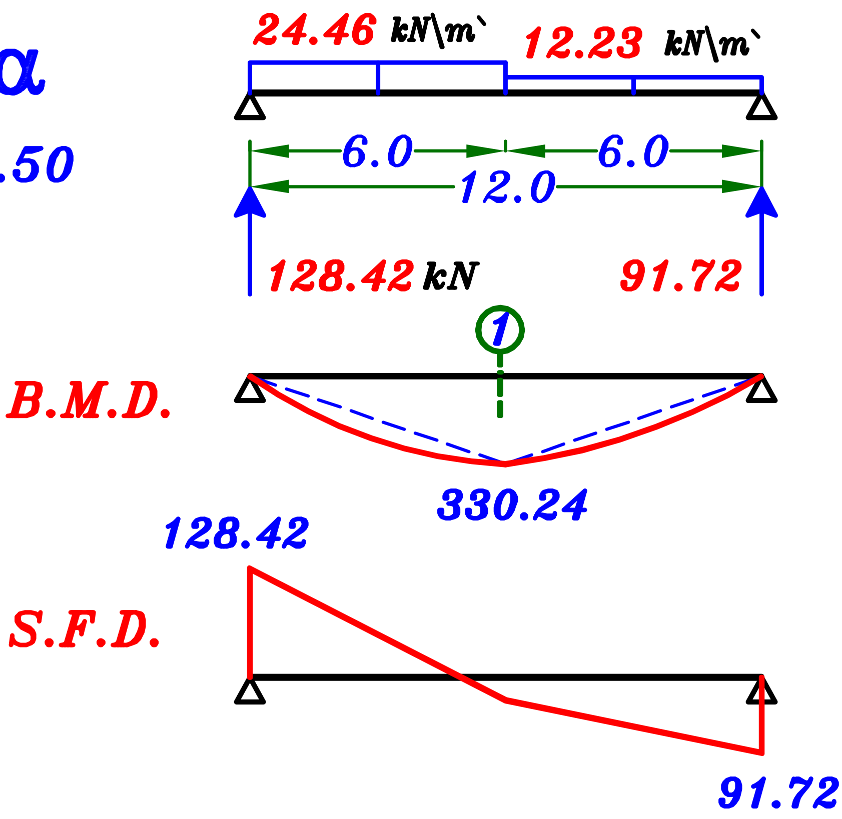
d- Get the Loads on the Panelled Beam & Calculate the B.M.



$a_1 = 3.0\ m \quad , \quad a_2 = 1.5\ m$

$$w_1 = w_{av.} * a_1 * \alpha$$
$$= 16.31 * 3.0 * 0.50$$
$$= 24.46\ kN \backslash m`$$

$$w_1` = w_{av.} * a_2 * \alpha$$
$$= 16.31 * 1.5 * 0.50$$
$$= 12.23\ kN \backslash m`$$



e - Calculate the reduction Factor of the B.M. $\left(\frac{Sin\ \theta}{Sin\ 90}\right)$

$X = 3.0\ m \quad , \quad \frac{L}{2} = 6.0\ m \qquad \theta_{B_1} = \frac{3.0}{6.0} * 90^\circ = 45^\circ$

$$M_1 = 330.24 * \frac{\sin 45^\circ}{\sin 90^\circ} = 233.5\ kN.m$$

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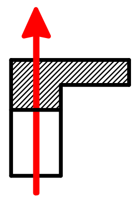
F- Design the Panelled Beam. B_1

$\therefore$ Cover = 70 mm   Symmetric



$t = 750\ mm \qquad d = 750 - 70 = 680\ mm$

$$B = \begin{cases} C.L.-C.L. = 1.5\ m = 1500\ mm \\ 6\ t_s + b = 6 * 140 + 250 = 1090\ mm \\ K\dfrac{L}{10} + b = 1.0 * \dfrac{12000}{10} + 250 = 1450\ mm \end{cases}$$

$$\boxed{B = 1090\ mm}$$

$$680 = C_1\sqrt{\frac{233.5*10^6}{25*1090}} \longrightarrow C_1 = 7.34 \longrightarrow J = 0.826$$

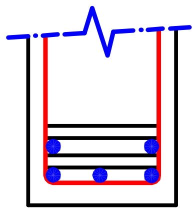
$$A_s = \frac{233.5*10^6}{0.826*360*680} = 1154.7\ mm^2$$

Check $A_{s\,min.}$ $A_{s_{req.}} = 1154.7\ mm^2$

$$\mu_{min.}\,b\,d = \left(0.225*\frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225*\frac{\sqrt{25}}{360}\right) 250*680 = 531.2\ mm^2$$

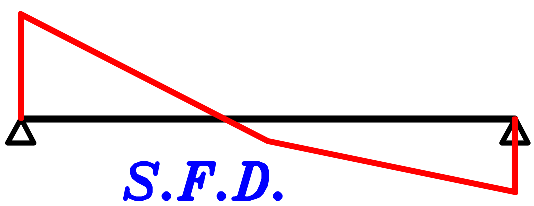
$\therefore A_{s_{req.}} > \mu_{min.}\,b\,d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1154.7\ mm^2$ $5\phi18$

$$n = \frac{b-25}{\phi+25} = \frac{250-25}{18+25} = 5.23 = 5.0\ bars$$



Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (115 \rightarrow 230\ mm^2)$ $3\phi10$

Check Shear.



$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98\ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85\ N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\,d} = \frac{128.42*10^3}{250*680} = 0.755\ N\backslash mm^2 \qquad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $5\phi8\backslash m'$

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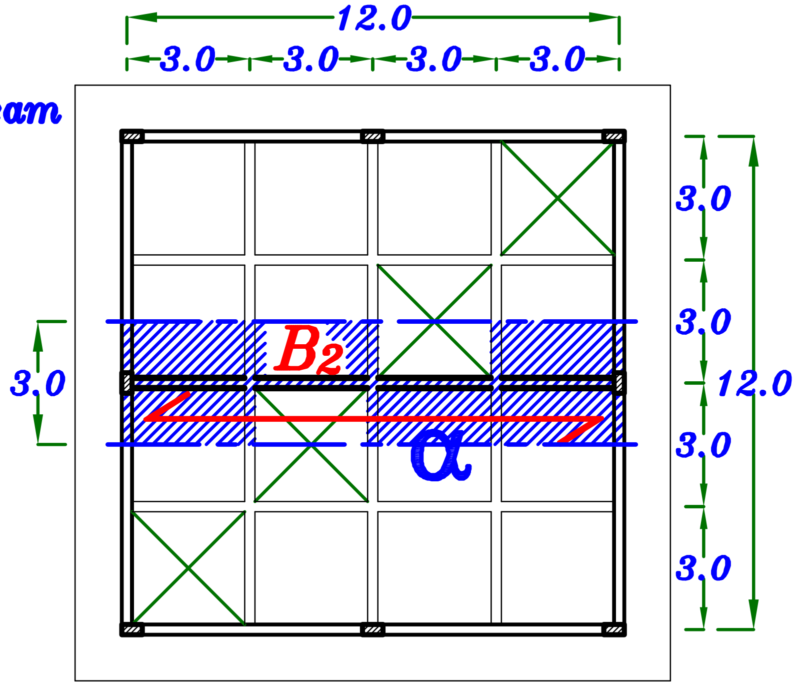
B_2 α Direction.

d- Get the Loads on the Panelled Beam & Calculate the B.M.



$$a_1 = 3.0\ m \quad , \quad a_2 = 1.5\ m$$

$$w_2 = w_{av.} * a_1 * \alpha$$

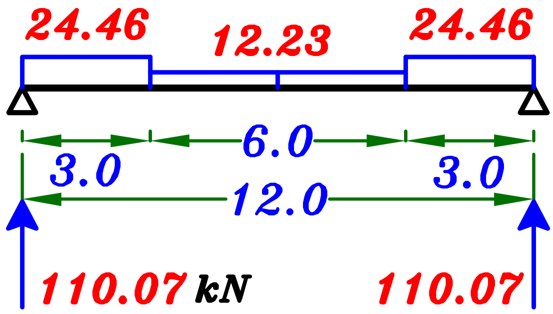
$$= 16.31 * 3.0 * 0.50$$

$$= 24.46\ kN\backslash m`$$

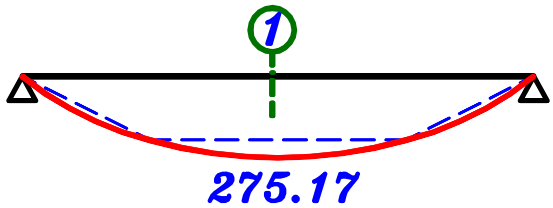
$$w_2` = w_{av.} * a_2 * \alpha$$

$$= 16.31 * 1.5 * 0.50$$

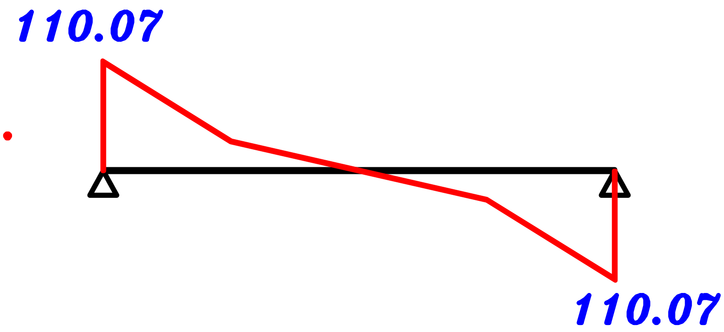
$$= 12.23\ kN\backslash m`$$



B.M.D.



S.F.D.



e - Calculate the reduction Factor of the B.M. $\left(\frac{Sin\ \theta}{Sin\ 90}\right)$

$$X = 6.0\ m \quad , \quad \frac{L}{2} = 6.0\ m \qquad \theta_{B_1} = \frac{6.0}{6.0} * 90^\circ = 90^\circ$$

$$M_1 = 275.17 * \frac{\sin 90^\circ}{\sin 90^\circ} = 275.17\ kN.m$$

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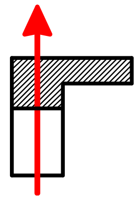
F- Design the Panelled Beam. B_2

$\therefore$ Cover $= 70$ mm Symmetric

$t = 750$ mm $\qquad d = 750 - 70 = 680$ mm



L – Sec.

$$B = \begin{cases} C.L.-C.L. = 1.5\ m = 1500\ mm \\ 6\ t_s + b = 6 * 140 + 250 = 1090\ mm \\ K\dfrac{L}{10} + b = 1.0 * \dfrac{12000}{10} + 250 = 1450\ mm \end{cases}$$

$$\boxed{B = 1090\ mm}$$

$$680 = C_1\sqrt{\frac{275.17 * 10^6}{25 * 1090}} \longrightarrow C_1 = 6.76 \longrightarrow J = 0.826$$

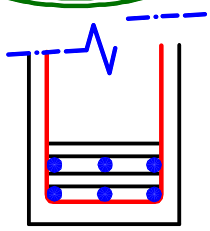
$$A_s = \frac{275.17 * 10^6}{0.826 * 360 * 680} = 1360.8\ mm^2$$

Check $A_{s\,min.}$ $\qquad A_{s\,req.} = 1360.8\ mm^2$

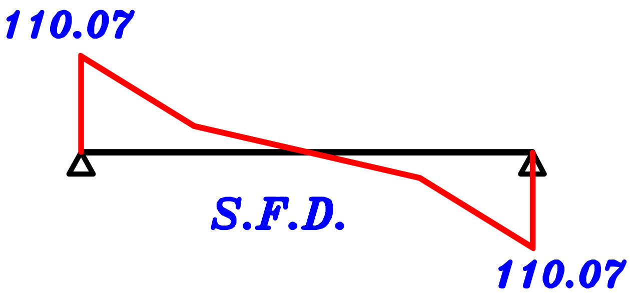
$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 680 = 531.2\ mm^2$$

$\therefore A_{s\,req.} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s\,req.} = 1360.8\ mm^2$ $\quad 6\phi18$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (136 \rightarrow 272\ mm^2)$ $\quad 3\phi10$



Check Shear.



S.F.D.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98\ N\backslash mm^2$$

$$q_{u\,max} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85\ N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\,d} = \frac{110.07 * 10^3}{250 * 680} = 0.64\ N\backslash mm^2 \qquad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $\quad 5\phi8\backslash m'$

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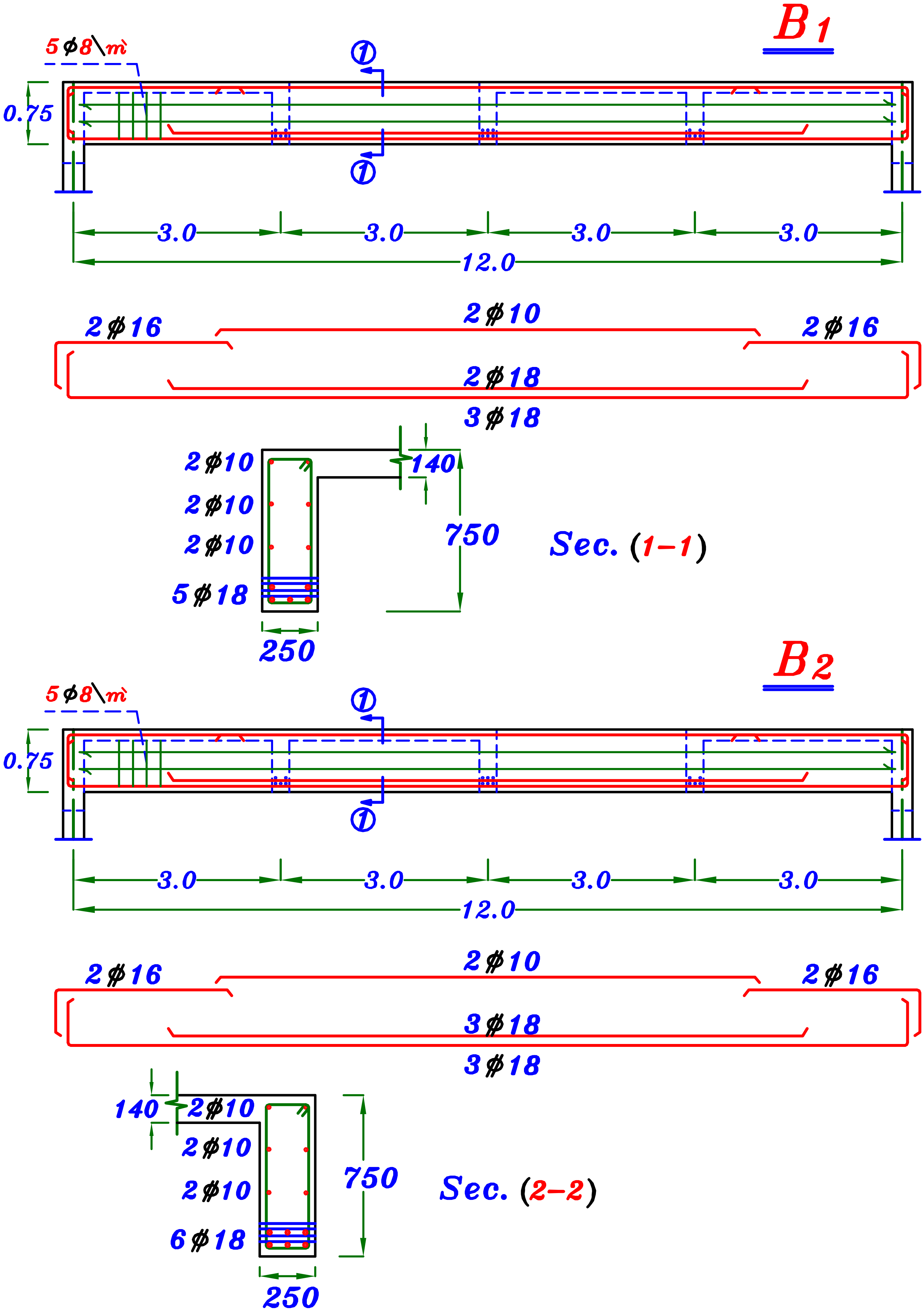
Draw Details of RFT. For the Beams. (B_1, B_2)



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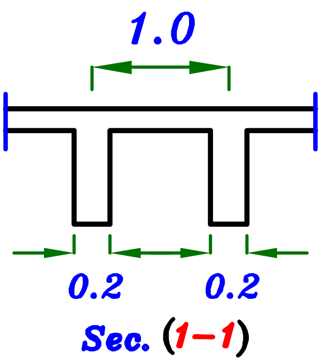
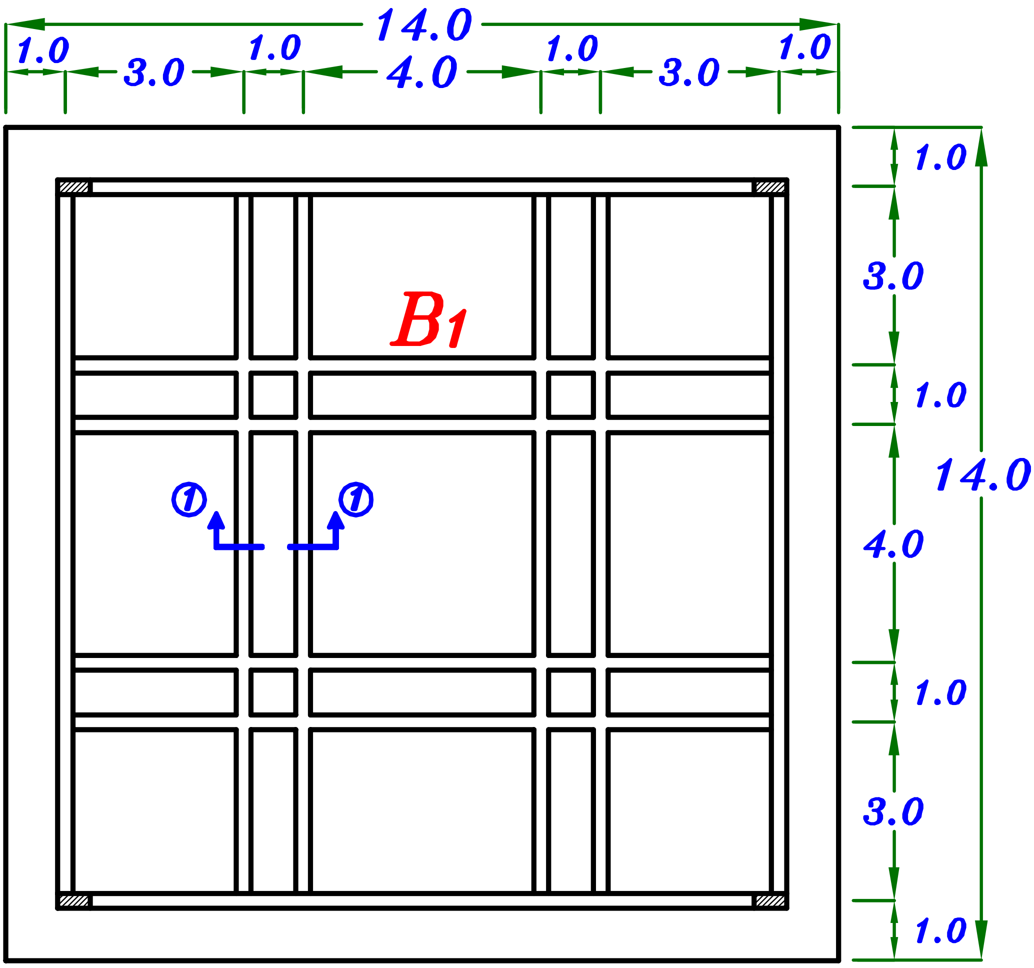
Example.





Data.

$F_{cu} = 25 \ N\backslash mm^2$ $F_y = 360 \ N\backslash mm^2$

$F.C. + L.L. = 3.0 \ kN\backslash m^2$

Req.

1. Design the Slabs as Solid Slabs.
2. Draw Details of RFT. of Slabs in plan.
3. Design Beam B_1
4. Draw Details of RFT. of the beam in elevation & cross sections.

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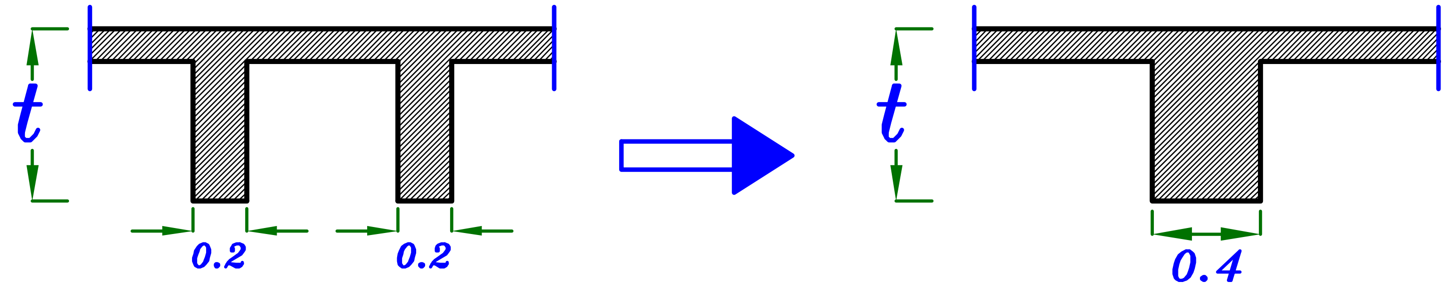
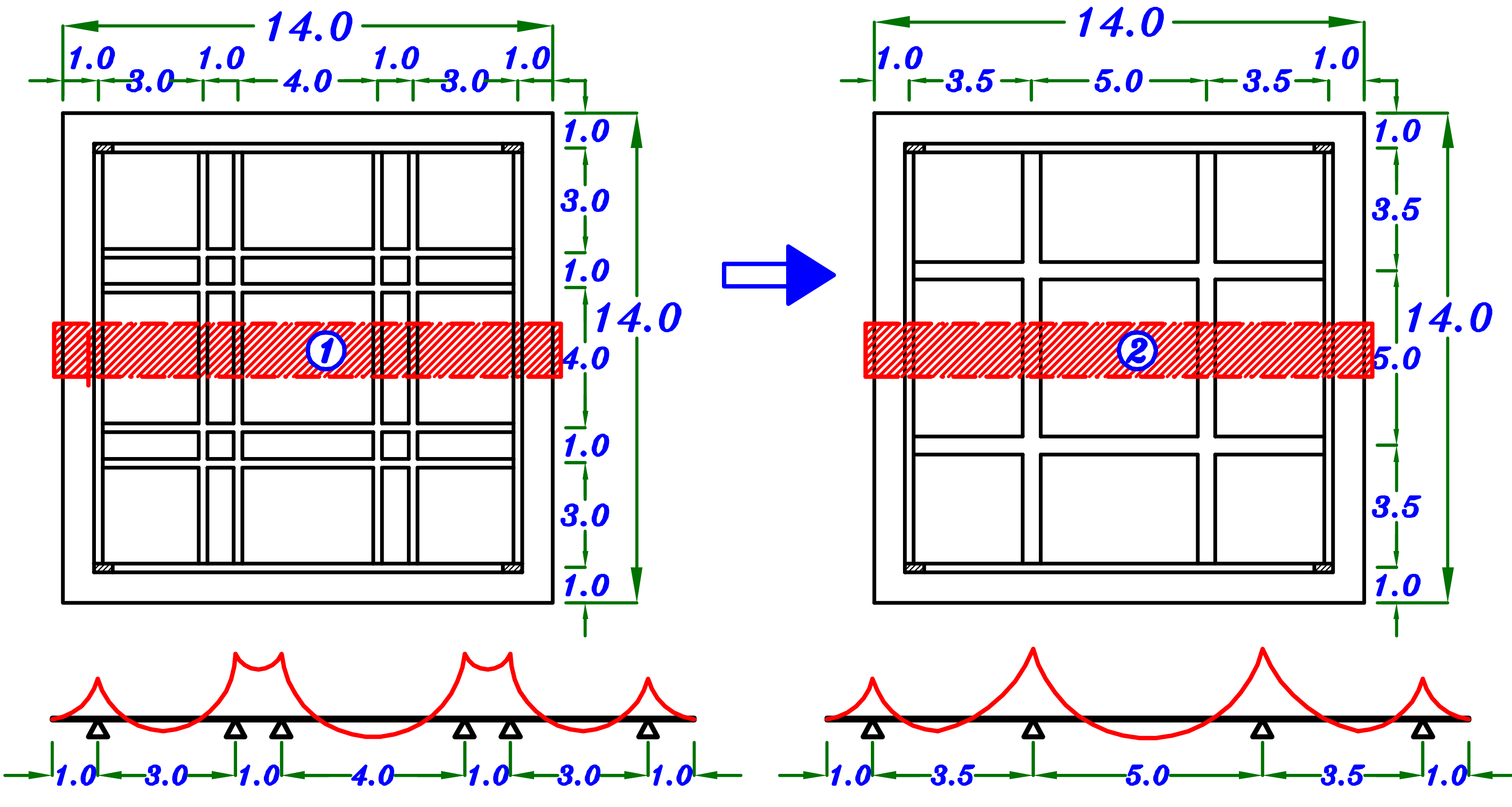
فكره المسأله .

ممكن للتسهيل عند حساب الاحمال و تصميم كلا من البلاطه و الكمره اعتبار انه تم ضم الكمرتين المتجاورتين معا و اعتبارهم فى الحسابات كأنها كمره واحده .



Sec. (1-1) Sec. (1-1)

و ذلك أثناء حسابات الاحمال و التصميم فقط لكن عند رسم التسليح يجب ان نسلح على الشكل الاصلى . **لان هذا ما سيتم تنفيذه فى الحقيقه**



Strip ① Strip ②

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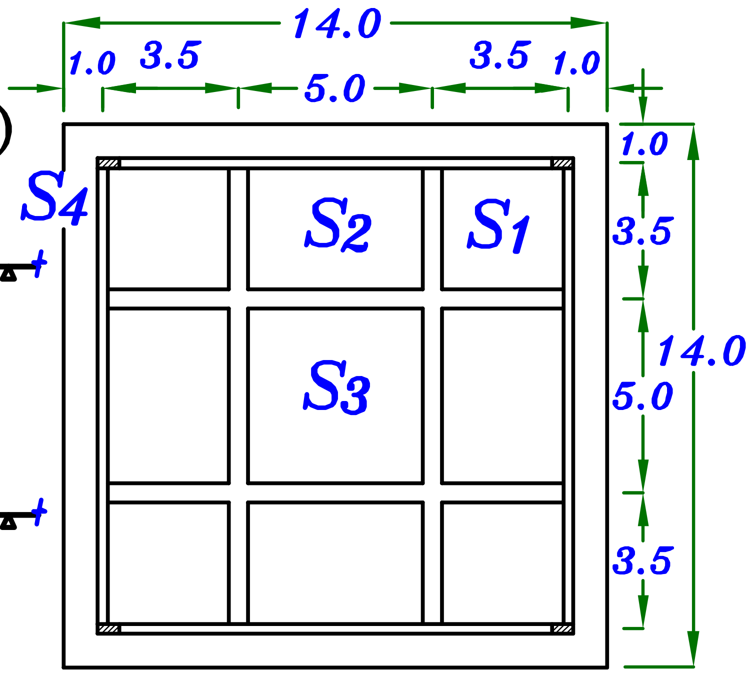
① Design the Slabs as Solid Slabs.



a- Choose the Thickness of the Slab. (t_s)

S_1 two way $L_s = 3.5\ m$

$$t_s = \frac{3500}{45} = 77.7\ mm$$

S_2 two way $L_s = 3.5\ m$

$$t_s = \frac{3500}{45} = 77.7\ mm$$

S_3 two way $L_s = 5.0\ m$

$$t_s = \frac{5000}{45} = 111.1\ mm$$

S_4 Cantilever $L_c = 1.0\ m$

$$t_s = \frac{1000}{10} = 100\ mm$$

Take (t_s) the bigger value $\boxed{t_s = 120\ mm}$

b- Get the Loads on the Slab (w_s).

$$\boxed{w_s = 1.5\,(t_s\,\gamma_c + F.C. + L.L.)}\quad kN\backslash m^2$$

$$w_s = 1.5\,(0.10 * 25 + 3.0) = 9.0\ kN\backslash m^2$$

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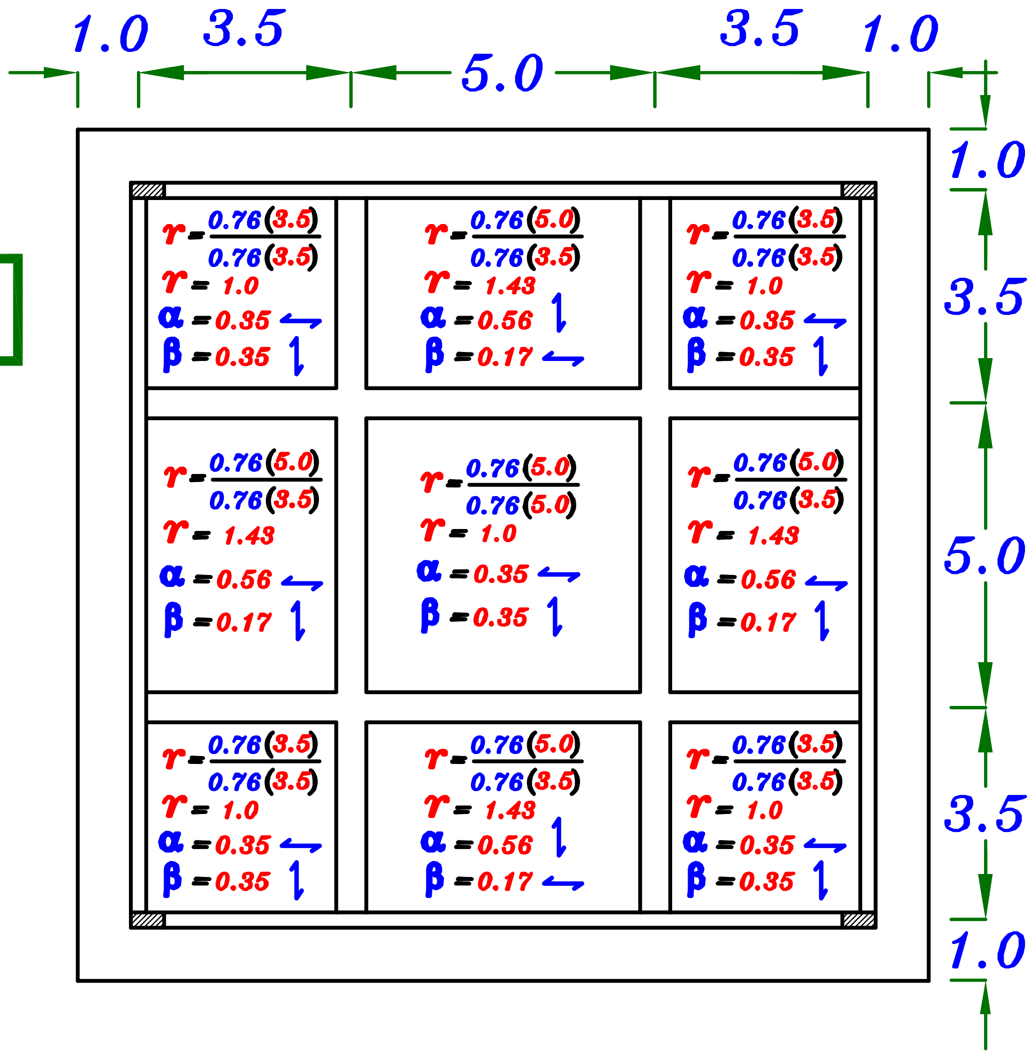
c - Get the Load Factors α, β

$$\alpha = 0.5\,\gamma - 0.15$$

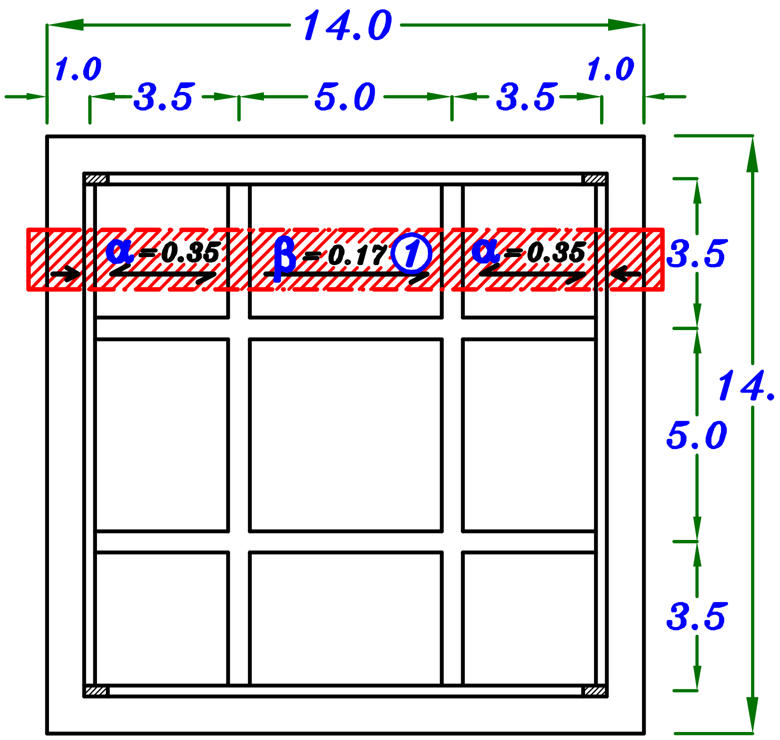
$$\beta = \frac{0.35}{\gamma^2}$$



d- Take a strips in the slab (at the Load direction)

And then Get (B.M.) on the Slab & Design the slab.

Strip ①



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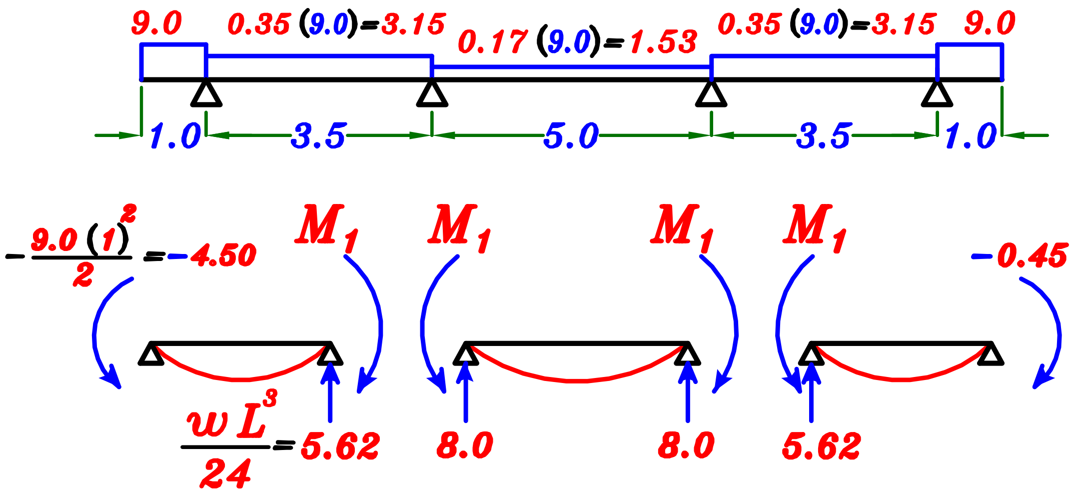
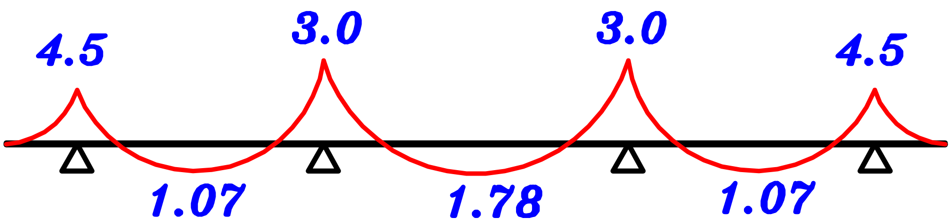
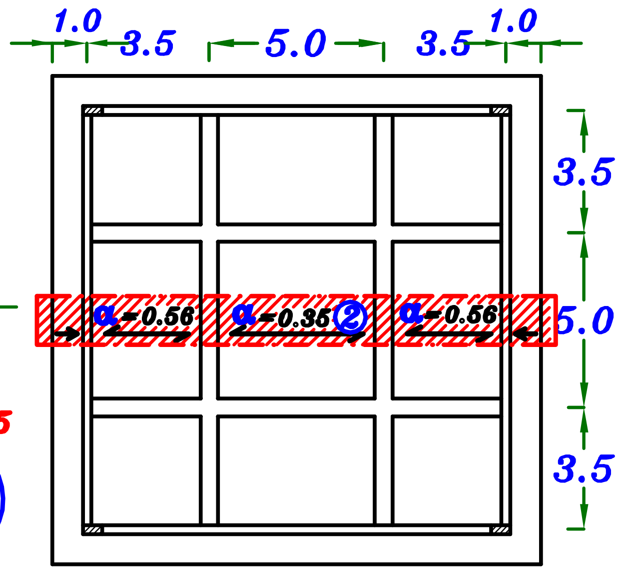


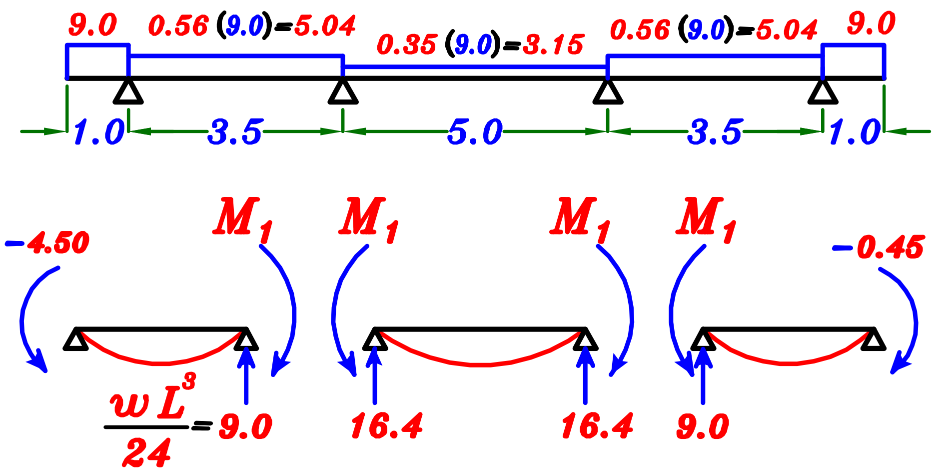
$$-4.5(3.5)+2M_1(3.5+5.0)+M_1(5.0)=-6\,(5.62+8.0)$$

$$\boxed{M_1=-\,3.0 \text{ kN.m}}$$



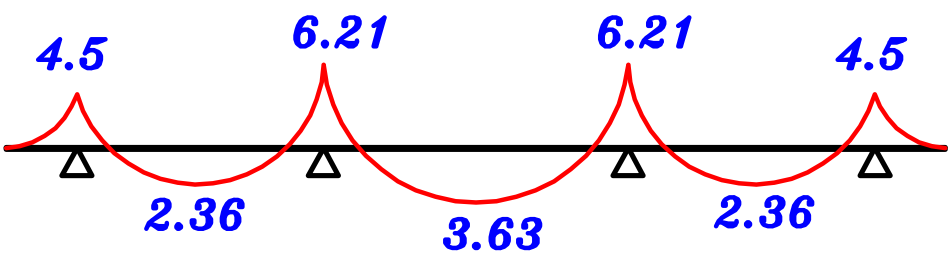
Strip 2





$$-4.5(3.5)+2M_1(3.5+5.0)+M_1(5.0)=-6\,(9.0+16.4)$$

$$\boxed{M_1=-\,6.21 \text{ kN.m}}$$



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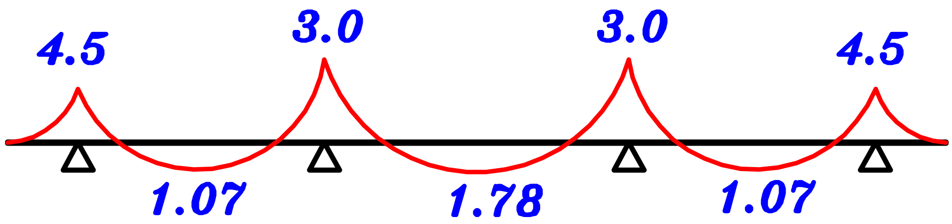
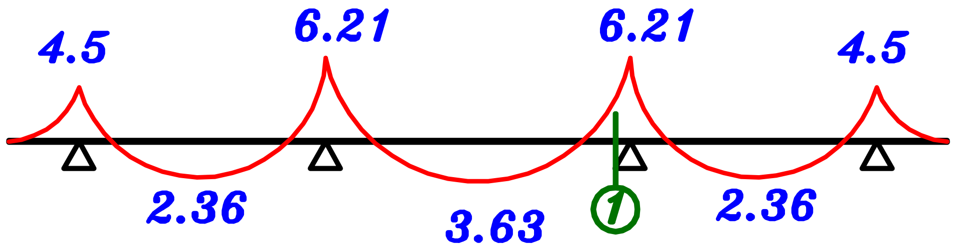
Strip ①



Strip ②



Sec. ① $M_{U.L.} = 6.21 \ kN.m\backslash m$

$t_s = 120 \ mm$, $d = 120 - 20 = 100 \ mm$, $B = 1000 \ mm$ عرض الشريحه

$$100 = C_1 \sqrt{\frac{6.21 * 10^6}{25 * 1000}} \longrightarrow C_1 = 6.34 \longrightarrow J = 0.826$$

$$A_s = \frac{6.21 * 10^6}{0.826 * 360 * 100} = 208.8 \ mm^2/m' \qquad \boxed{5\phi 10 \backslash m'}$$

$\boxed{\therefore}$ سيؤخذ تسليح باقى القطاعات $5\phi 10 \backslash m'$

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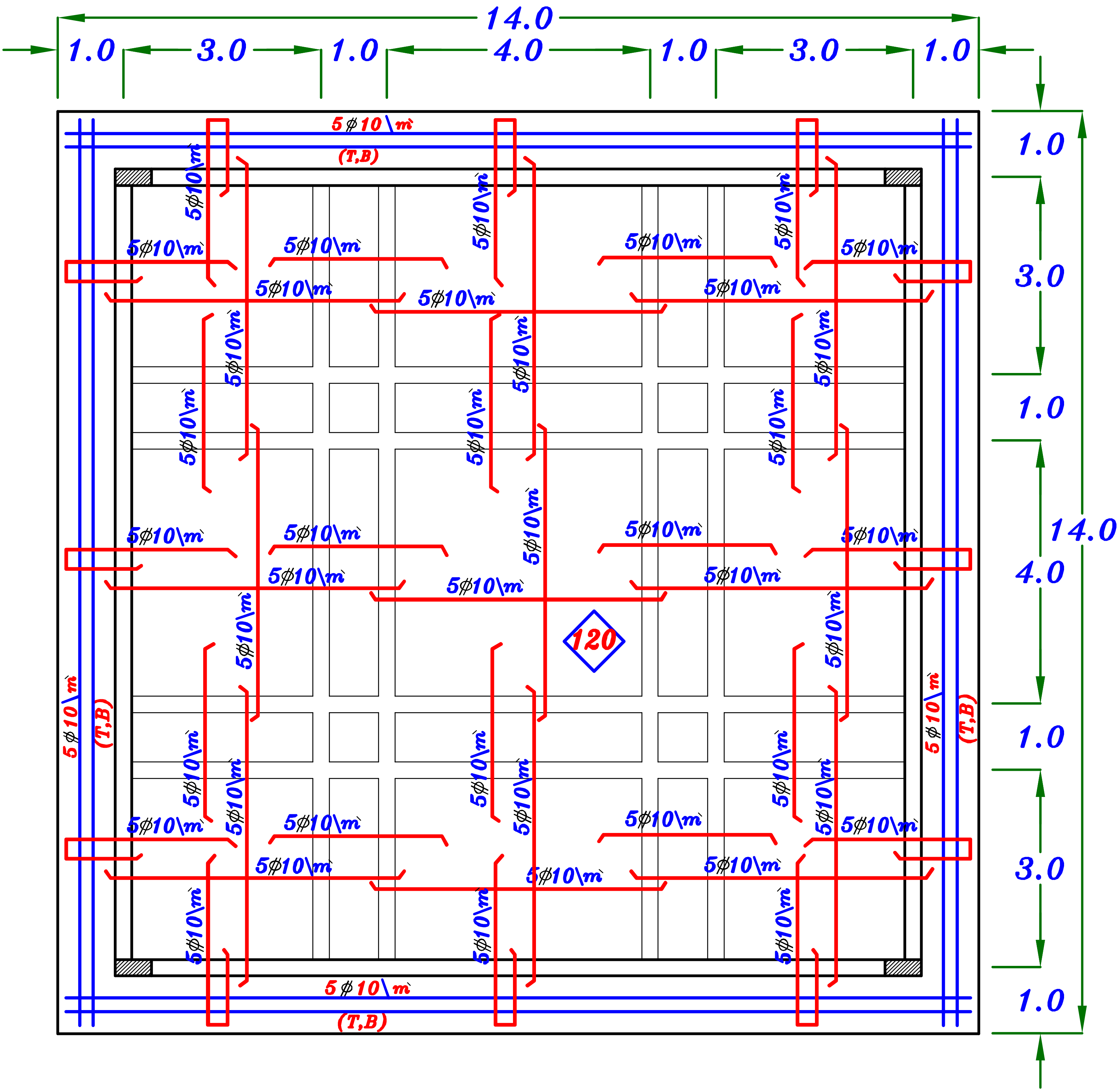
Details of RFT. For the Slab.



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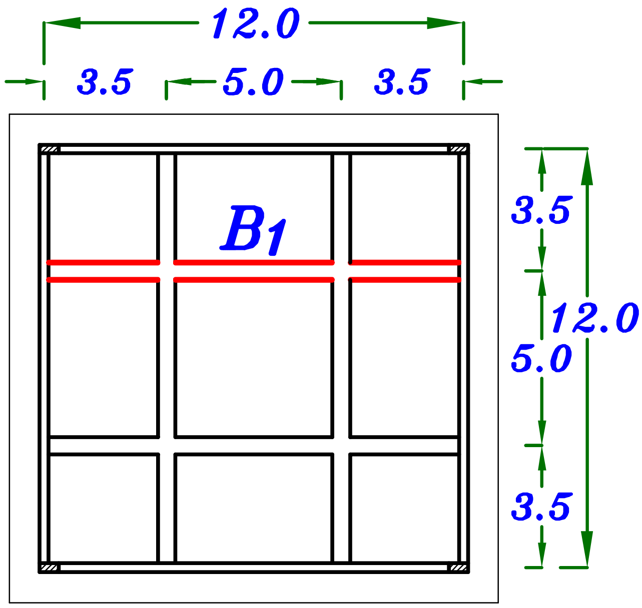
F- Design the Panelled Beam. B_1

a- Get the Dimensions of the beam. (b, t)



Take $b = 0.40\ m$

as Ginev in data

$$t = \frac{L_s}{16} = \frac{12.0}{16} = 0.75\ m$$

$$\boxed{b = 0.25\ m} \qquad \boxed{t = 0.75\ m}$$

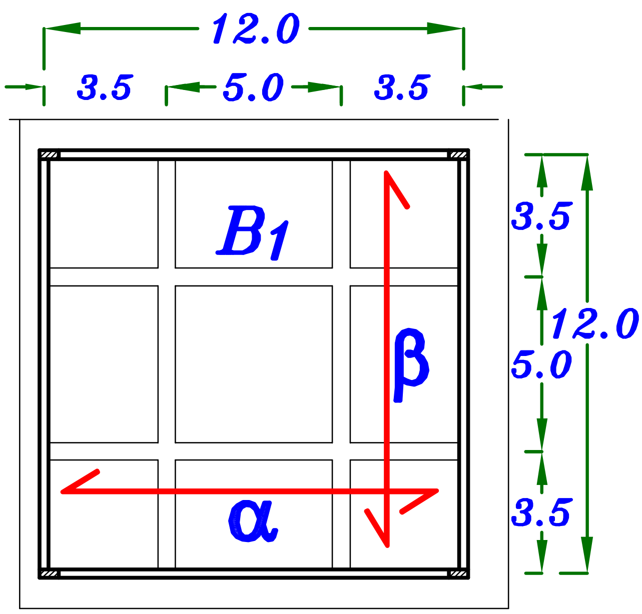
b- Get the Loads on the Slab. (w_{av})

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s}$$

$$w_{av.} = w_s + \frac{1.4 * b (t - t_s) \left[\text{مجموع أطوال الكمرات الداخلية}\right] * \gamma_c}{L * L_s}$$

$$w_{av.} = 9.0 + \frac{1.4 * 0.40 (0.75 - 0.12) \left[2*12 + 2*12\right] * 25}{12 * 12} = 11.94\ kN\backslash m^2$$

c- Calculate α, β By using Grashoff.



$$r = \frac{m\ L}{m'\ L_s} = \frac{(1.0)\ 9.0}{(1.0)\ 9.0} = 1.0$$

$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.0)^4}{1 + (1.0)^4} = 0.50$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.0)^4} = 0.5$$

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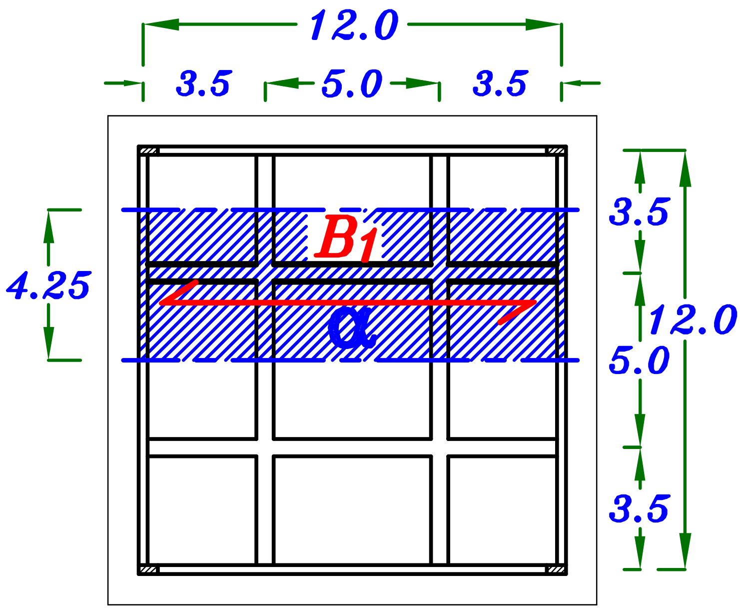
B_1 α Direction.



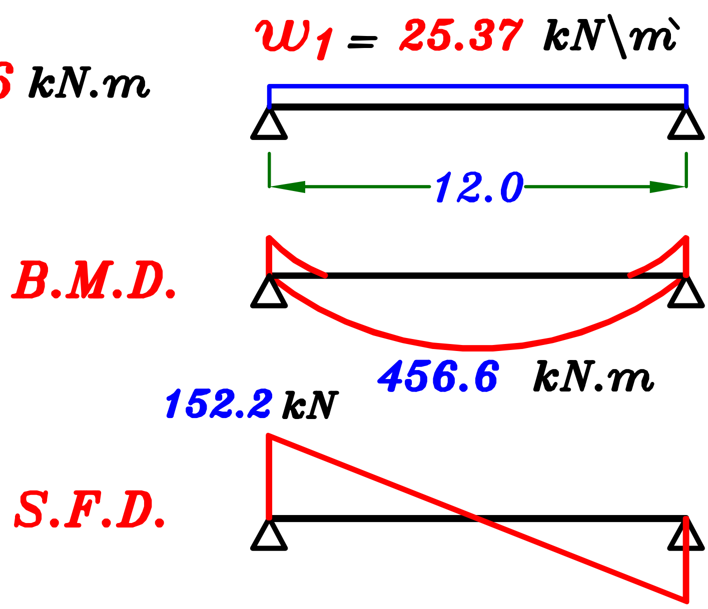
$$a = \frac{3.5}{2} + \frac{5.0}{2} = 4.25\ m$$

$$w_1 = w_{av.} * a * \alpha$$

$$= 11.94 * 4.25 * 0.50$$

$$= 25.37\ kN\backslash m'$$

$$M = 25.37 * \frac{12.0^2}{8} = 456.6\ kN.m$$



e- Calculate the reduction Factor of the B.M. $\left(\frac{Sin\ \theta}{Sin\ 90}\right)$

$$X = 3.5\ m\ ,\quad \frac{L}{2} = 6.0\ m$$

$$\theta_{B_1} = \frac{3.5}{6.0} * 90^\circ = 52.5^\circ$$

$$M_1 = 456.6 * \frac{\sin 52.5^\circ}{\sin 90^\circ} = 362.2\ kN.m$$

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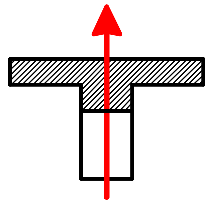
F– Design the Panelled Beam. B_1

فى الكمرات ال symmetric نصمم كمره واحده فقط و نضع تسليح الكمرتين مثل بعض و يفضل ان نأخذ cover كمره β لانه الاكبر لانه فى التصميم كلما فرضنا ان ال cover اكبر تقل قيمه d فى التصميم فتزيد كميه الحديد .

β Direction. $\therefore$ Cover $= 70$ mm

$t = 750$ mm $\quad d = 750 - 70 = 680$ mm



T– Sec.

$$B = \begin{cases} C.L.-C.L. = \frac{3.5}{2} + \frac{5.0}{2} = 4.25\ m = 4250\ mm \\ 16\, t_s + b = 16 * 120 + 400 = 2320\ mm \\ K\, \frac{L}{5} + b = 1.0 * \frac{12000}{5} + 400 = 2800\ mm \end{cases}$$

$$\boxed{B = 2320\ mm}$$

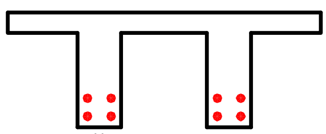
$$680 = C_1 \sqrt{\frac{362.2 * 10^6}{25 * 2320}} \longrightarrow C_1 = 8.60 \longrightarrow J = 0.826$$

$$A_s = \frac{362.2 * 10^6}{0.826 * 360 * 680} = 1791\ mm^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 1791\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 680 = 850\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1791\ mm^2$ $\quad$ 8 ϕ 18



Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (179 \rightarrow 358\ mm^2)$ $\quad$ 4 ϕ 10

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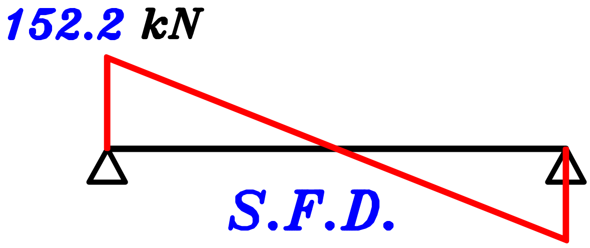
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

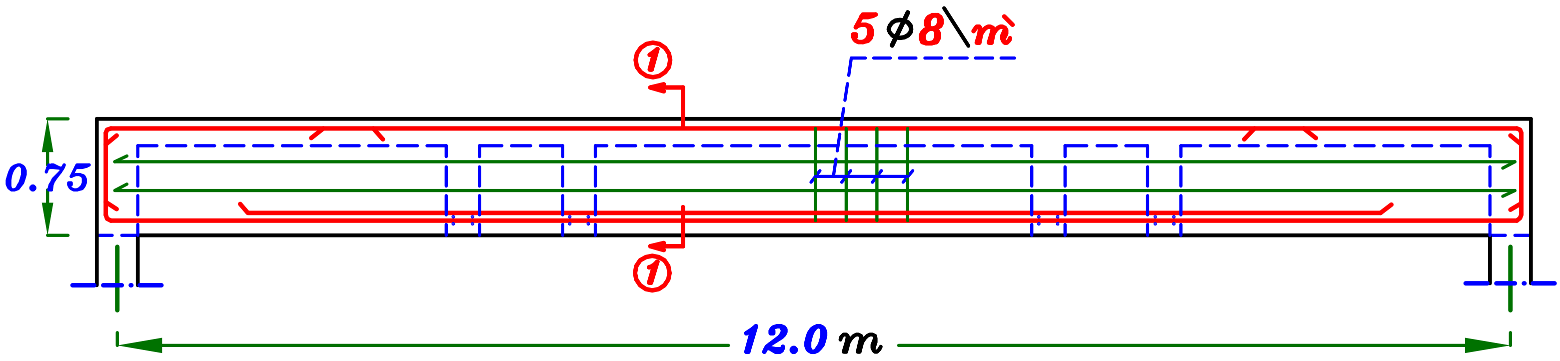
$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$

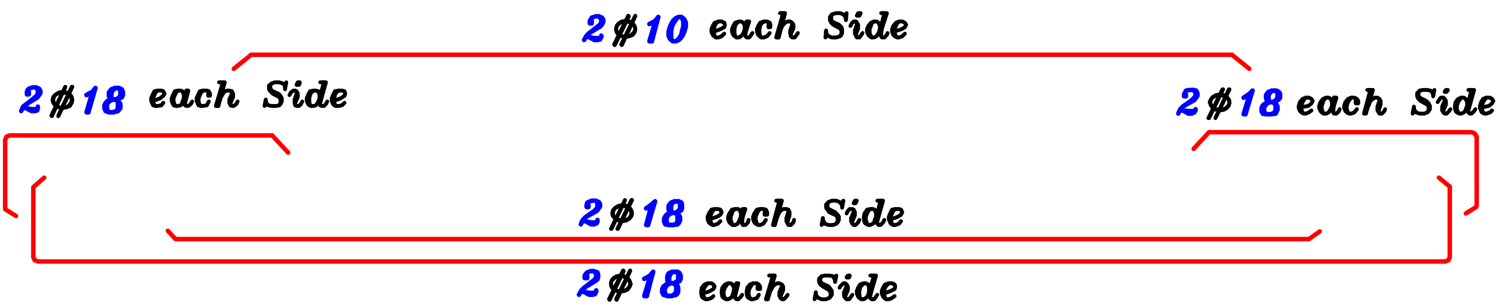


$$q_s = \frac{Q_{max}}{b\ d} = \frac{152.2 * 10^3}{250 * 680} = 0.89 \ N\backslash mm^2 \qquad \therefore q_s < q_{cu}$$

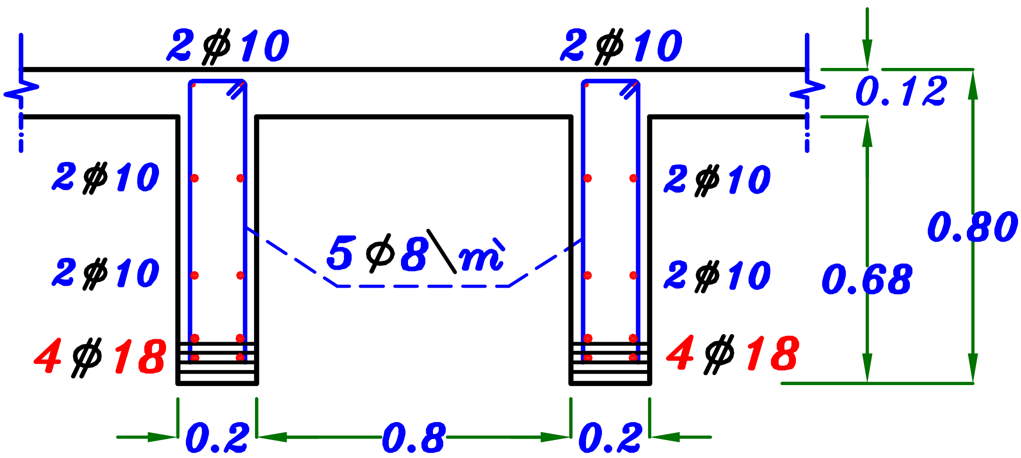
$\therefore$ Use min. Shear RFT. $\quad 5\phi 8\backslash m'$

Draw Details of RFT. For the Beams.





Sec. (1–1)



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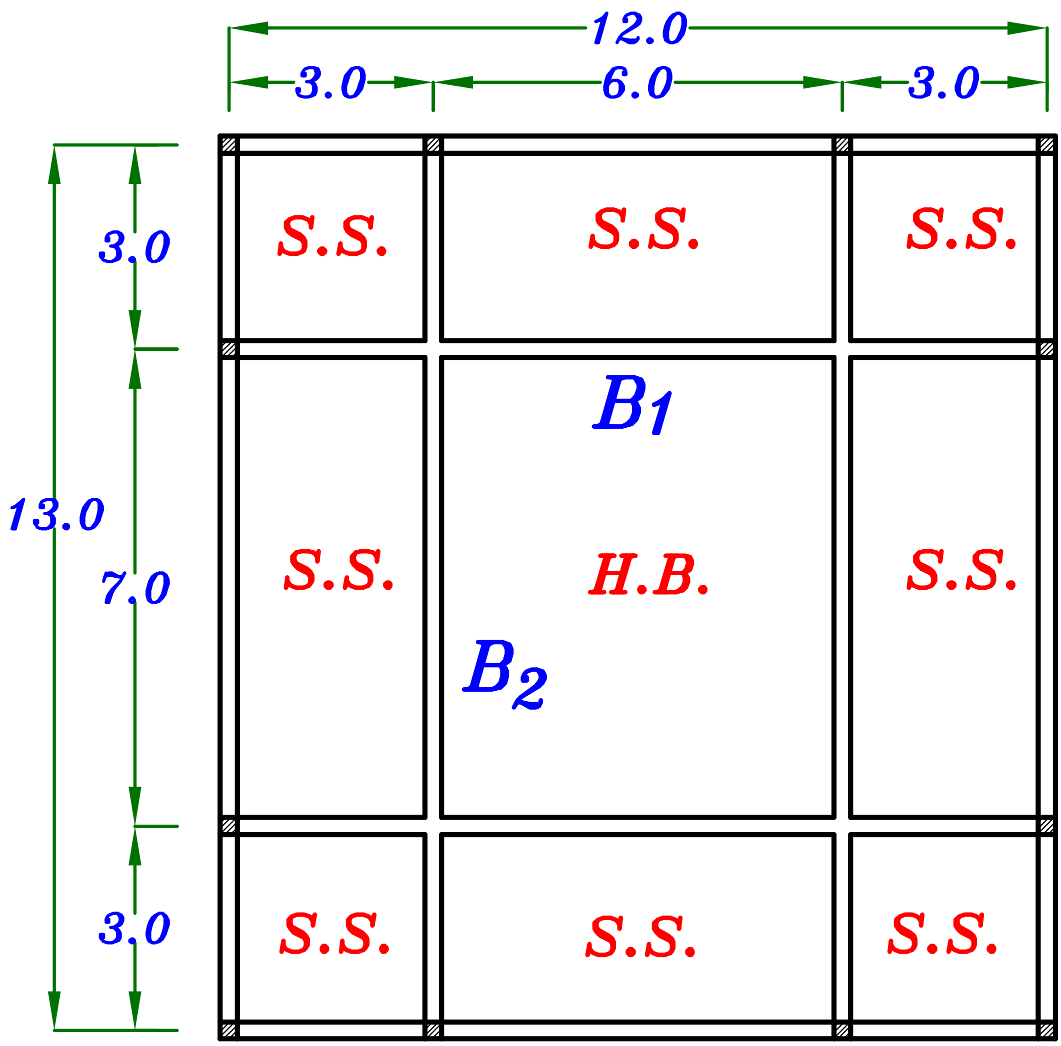
Example.



Data.

$F_{cu} = 25$ N\mm², $F_y = 360$ N\mm²

F.C. = 1.50 kN\m², L.L. = 2.0 kN\m²

Req.

1. Design the slab & Draw Details of RFT.
2. Design the panelled Beams B_1, B_2 & Draw Details of RFT. in elevation.

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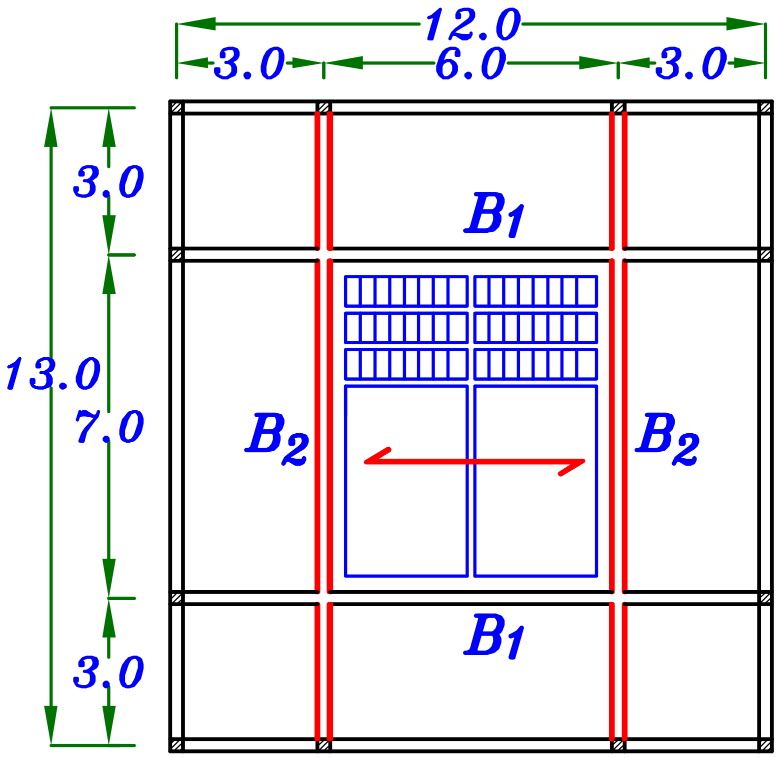


اذا اخذنا البلاطه $(6.0 * 7.0)$

عباره عن *One way H.B.*

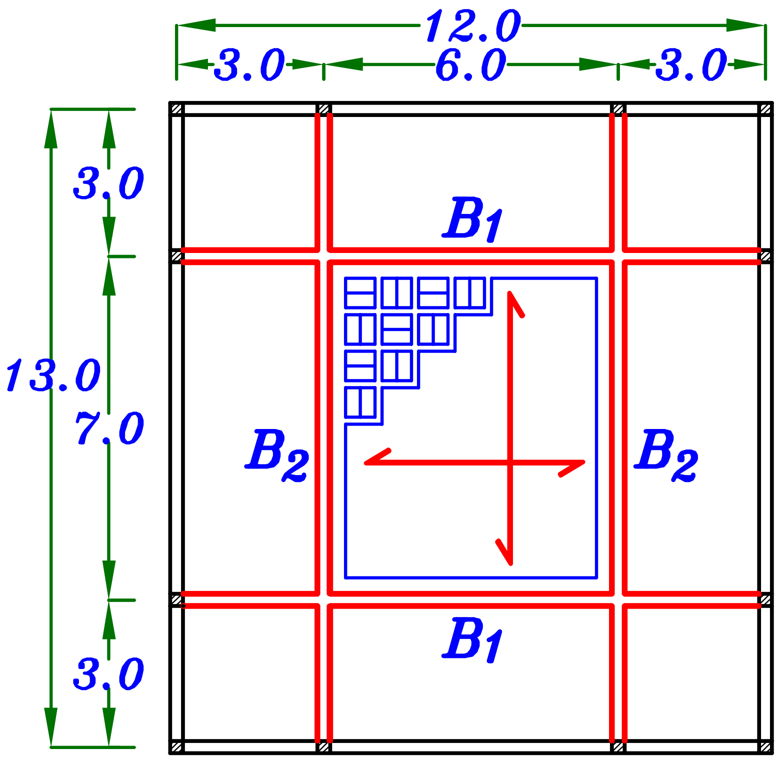
هذا معناه ان الكمرتين B_1

هما من سيحملان البلاطه كلها

و الكمرتان B_2 لن يحملا شياء من البلاطه

فستصبح شبكه الـ *Panelled Beams*

غير منتظمه و بذلك تفقد ميزاتها .



لذا سنضطر اخذ البلاطه فى المنتصف

Two way H.B.

حتى يتوزع حملها على الـ ٤ كمرات

خصوصا ان $\frac{L}{L_s} \ngtr \frac{4}{3}$

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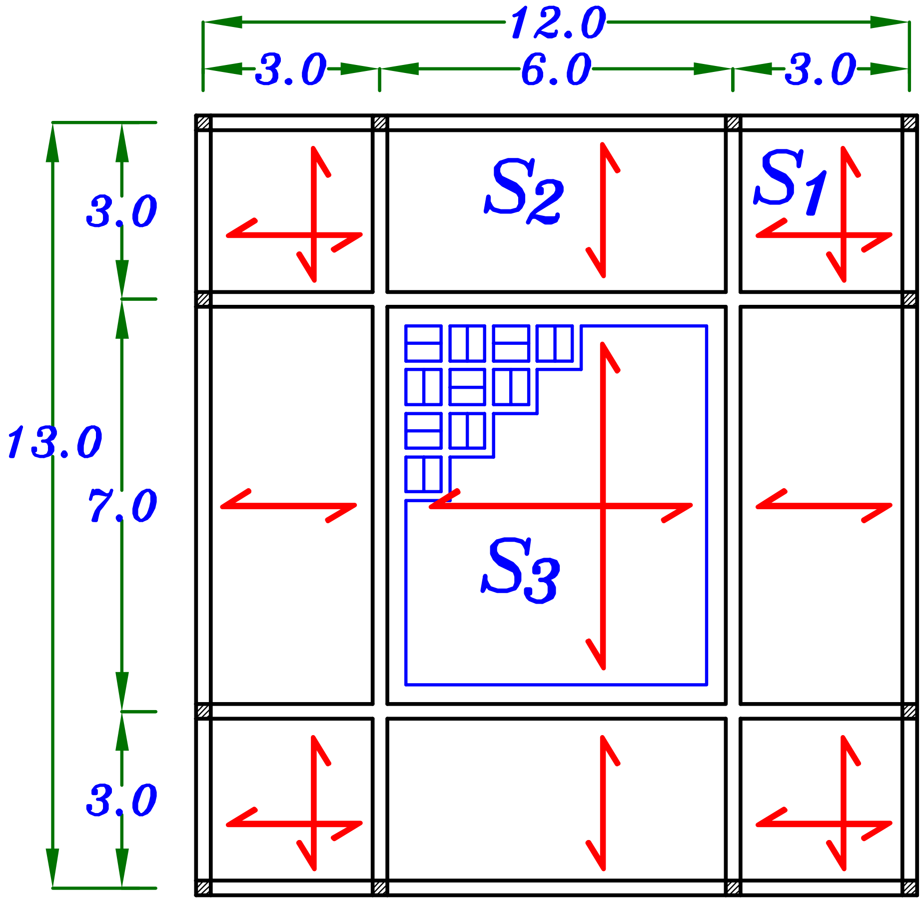
a- Choose the Thickness of the Slabs.



S_1 two way $L_s = 3.0\ m$ $\qquad t_s = \frac{3000}{40} = 75.0\ mm$

S_2 One way $L_s = 3.0\ m$ $\qquad t_s = \frac{3000}{30} = 100\ mm$

Take (t_s) the bigger value $\boxed{t_s = 100\ mm}$

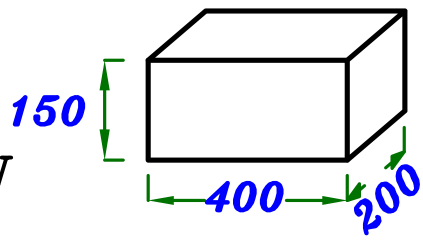
S_3 Two way H.B. $L_s = 6.0\ m$

$t = \frac{6000}{45} = 133\ mm$ $\qquad \boxed{t = 200\ mm}$

Take $\boxed{t = 200\ mm}$ $\quad \boxed{t_s = 50\ mm}$ $\quad \boxed{h = 150\ mm}$

The Block $(200 * 400 * 150)$

$h = 150\ mm \longrightarrow$ Weight of Block $= 100\ N$



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b_ Get the Loads on the Slab

For Solid Slabs.

$$W_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\ L.L.$$

$$W_s = 1.4\,(0.10 * 25 + 1.5) + 1.6\,(2.0) = 8.80 \quad kN\backslash m^2$$

For Two way Hollow Blocks.

$$\boxed{\begin{aligned} W_{ribT} &= \left[1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)\right](S * S) \\ &+ 1.4 * b\,h * (2S - b) * \gamma_c + 1.4 * (\text{Block وزن ال})\left(\frac{e}{a}\right) \end{aligned}}$$

$$\therefore W_{ribT} = \left[1.4\,(0.05 * 25 + 1.5) + 1.6\,(2.0)\right](0.5 * 0.5)$$

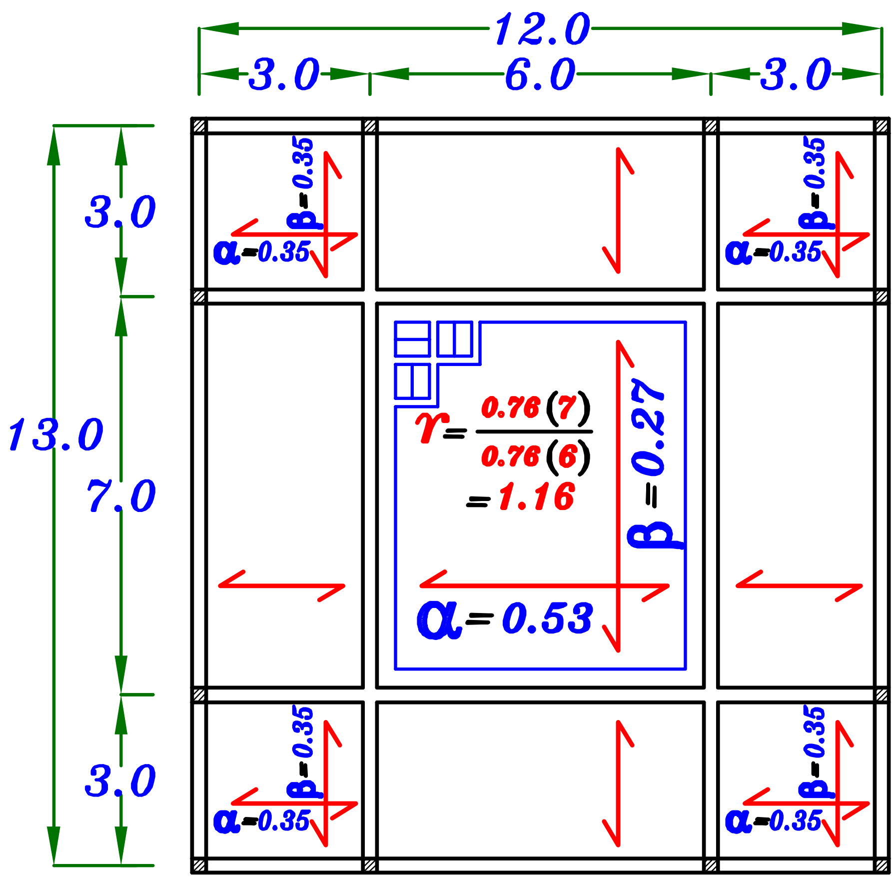
$$+ 1.4\,(0.1 * 0.15 * (2 * 0.5 - 0.1) * 25) + 1.4\left(\frac{100}{1000}\right)\left(\frac{0.4}{0.2}\right) = 2.515$$

$$(kN\backslash(S * S))$$

$$W_{rib} = \frac{W_{ribT}}{S} = \frac{2.515}{0.5} = 5.03 \quad kN\backslash(S * m)$$

For H.B.
Use Marcus

For S.S.
Use C.P.



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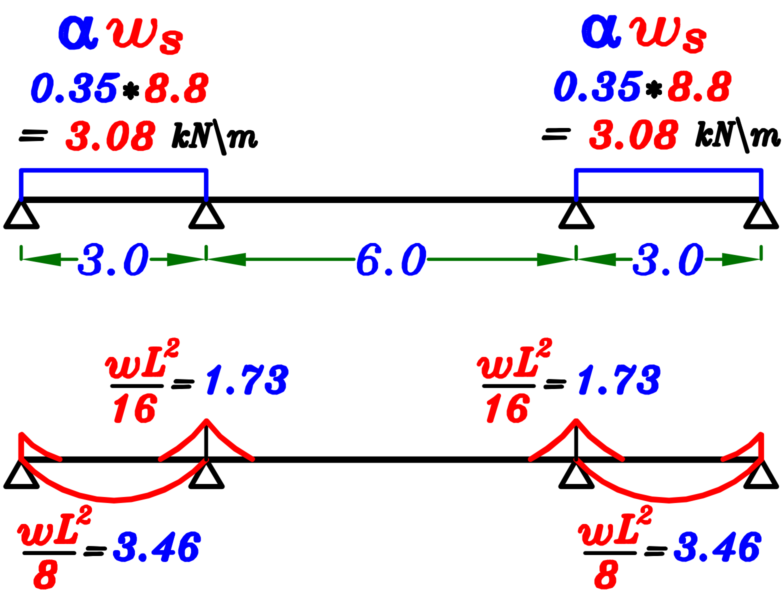
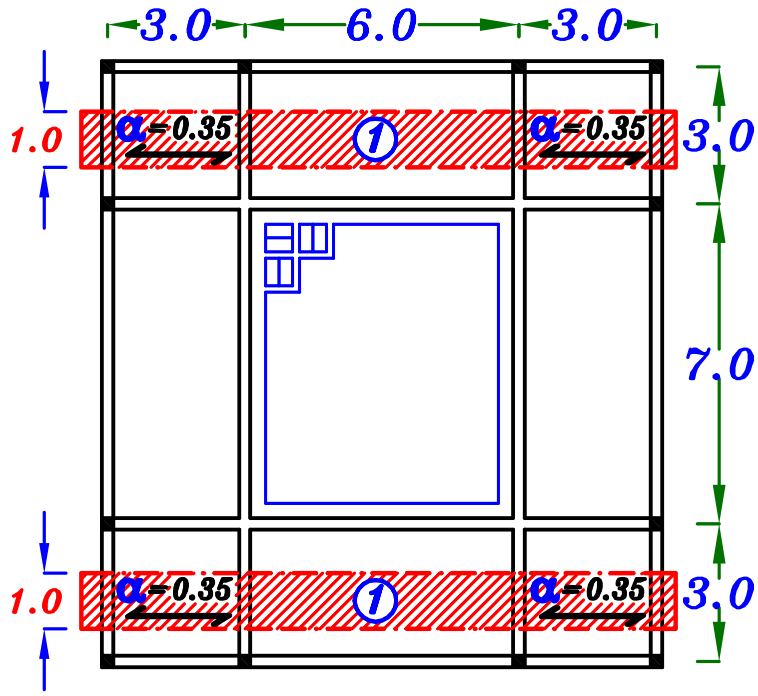
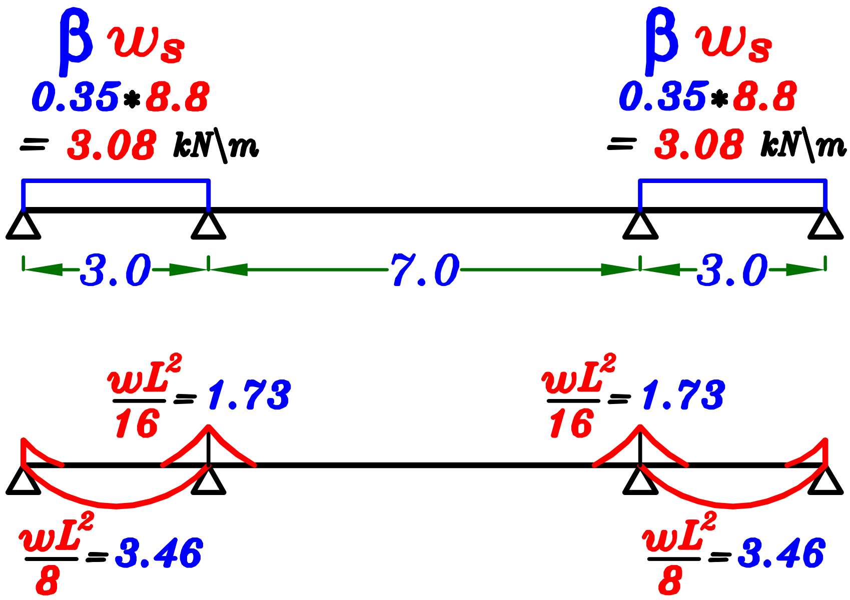
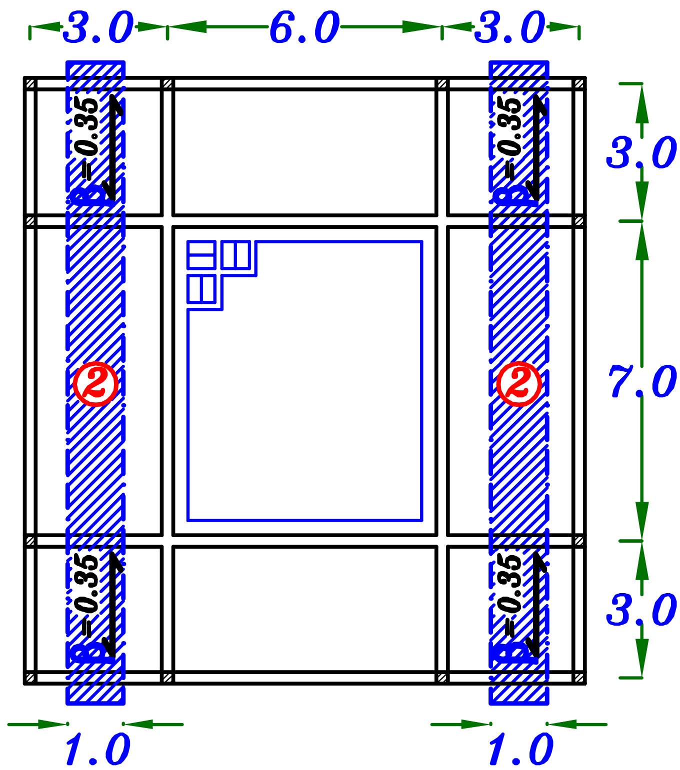
Strip ①





Strip ②





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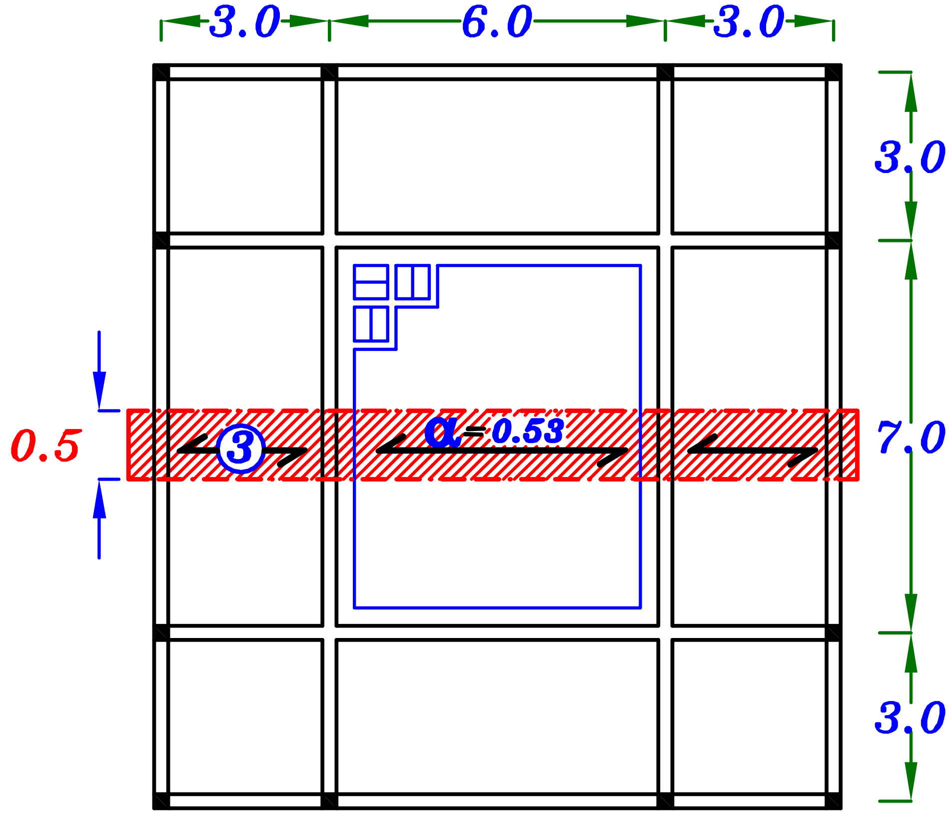
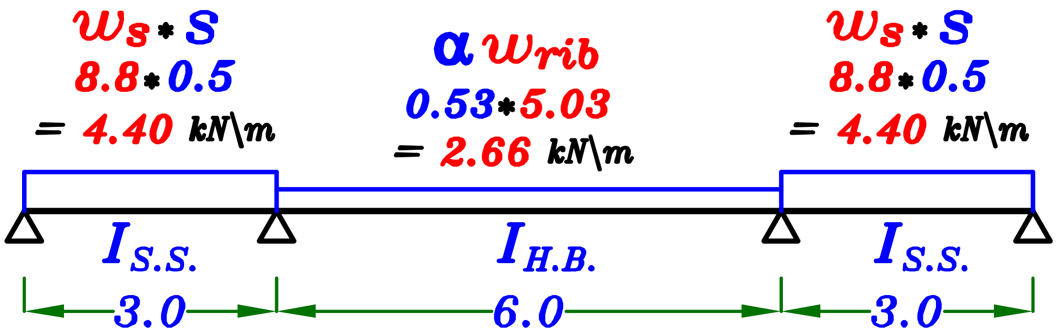
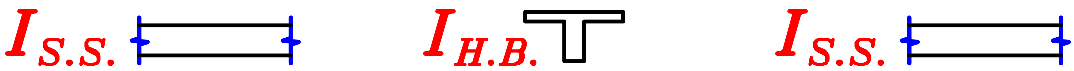
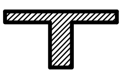
Strip ③



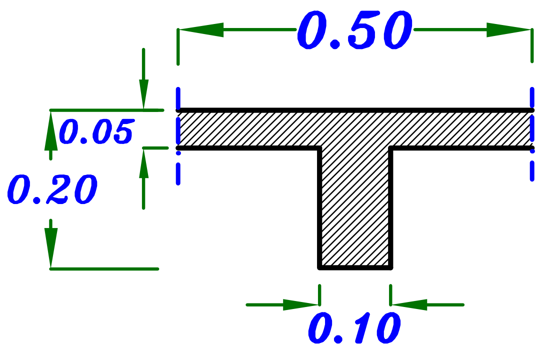




$I_{H.B.} =$  $\qquad I_1 = (\mu * 10^{-4})\ B\, t^3$

$B = 0.5\ m \quad , \quad t = 0.20\, m$

$\dfrac{t_s}{t} = \dfrac{0.05}{0.20} = 0.25 \ , \ \dfrac{b_o}{B} = \dfrac{0.1}{0.5} = 0.2$



From Tables page 91 $\quad \mu = 318$

$I_{H.B.} = (318 * 10^{-4} * 0.5 * 0.20^3) = 1.27 * 10^{-4}\ m^4$

$I_{S.S.} = \dfrac{S\,(t_s)^3}{12} = \dfrac{0.5\,(0.10)^3}{12} = 4.16 * 10^{-5}\ m^4$

$\therefore \dfrac{I_{H.B.}}{I_{S.S.}} = \dfrac{1.27 * 10^{-4}}{4.16 * 10^{-5}} = 3.05 \qquad \therefore \boxed{I_{H.B.} = 3.05\ I_{S.S.}}$

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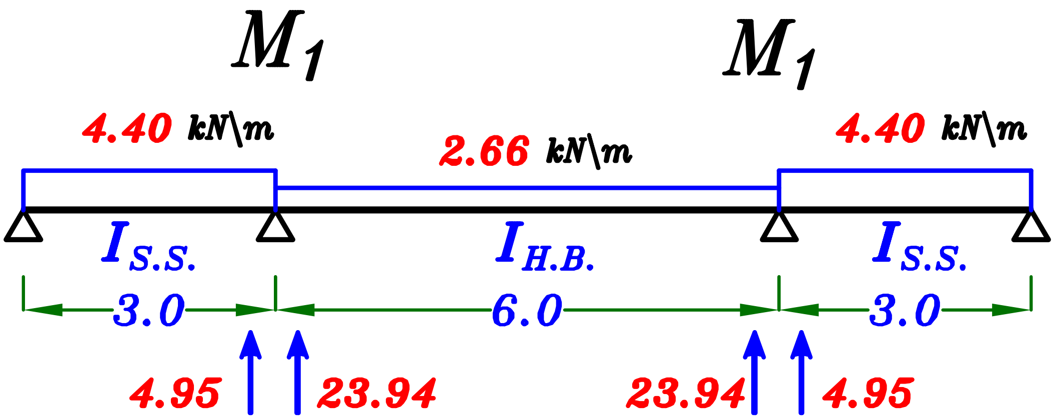
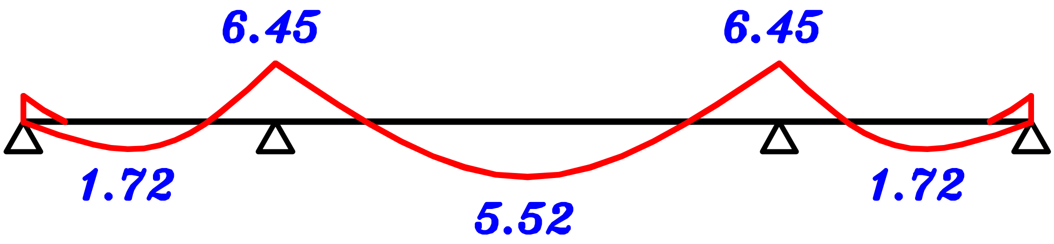
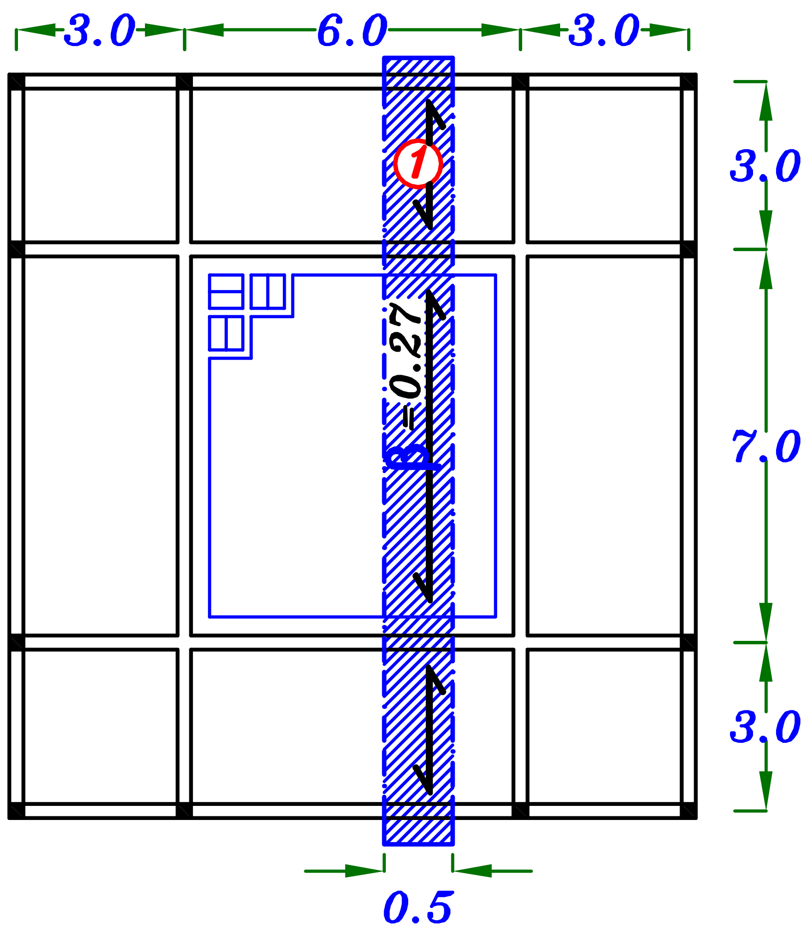


$$0.0 + 2\,M_1\left(\frac{3.0}{I_{s.s.}} + \frac{6.0}{3.05\,I_{s.s.}}\right) + M_1\left(\frac{6.0}{3.05\,I_{s.s.}}\right) = -6\left(4.95 + \frac{23.94}{3.05\,I_{s.s.}}\right)$$

$$\boxed{M_1 = -\,6.45 \ kN.m\backslash 0.5\,m}$$



Strip ④



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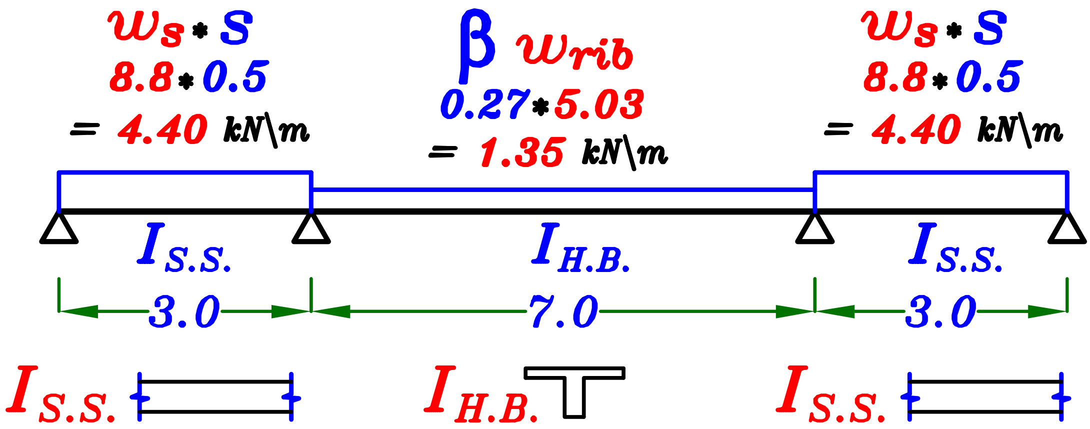
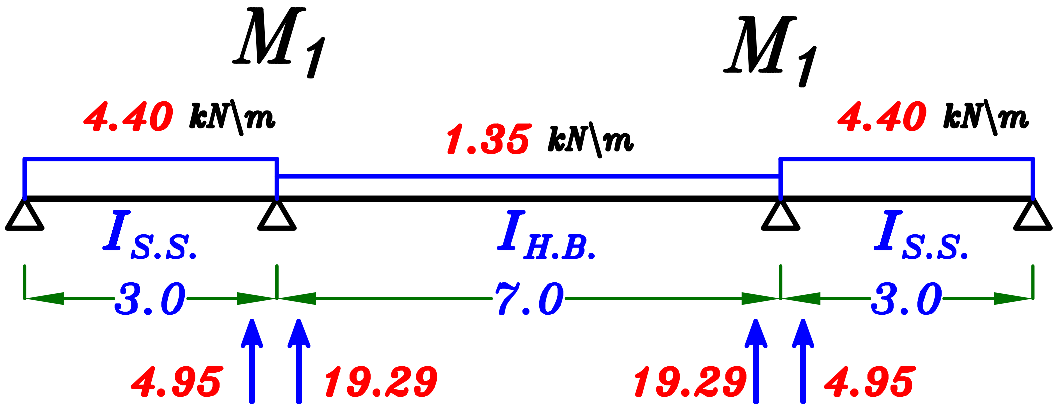
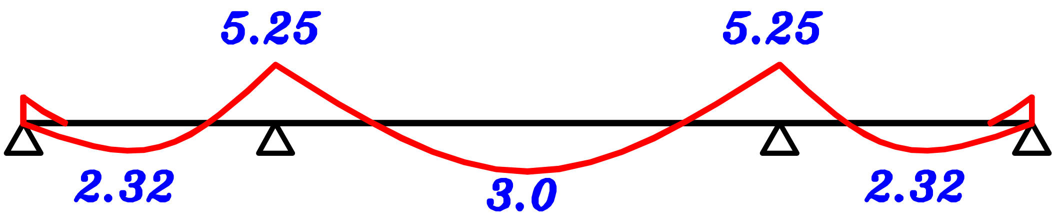


$$\boxed{I_{H.B.} = 3.05 \; I_{S.S.}}$$



$$0.0 + 2M_1\left(\frac{3.0}{I_{s.s.}} + \frac{7.0}{3.05 I_{s.s.}}\right) + M_1\left(\frac{7.0}{3.05 I_{s.s.}}\right) = -6\left(4.95 + \frac{19.29}{3.05 I_{s.s.}}\right)$$

$$\boxed{M_1 = -5.25 \;\; kN.m\backslash 0.5\,m}$$



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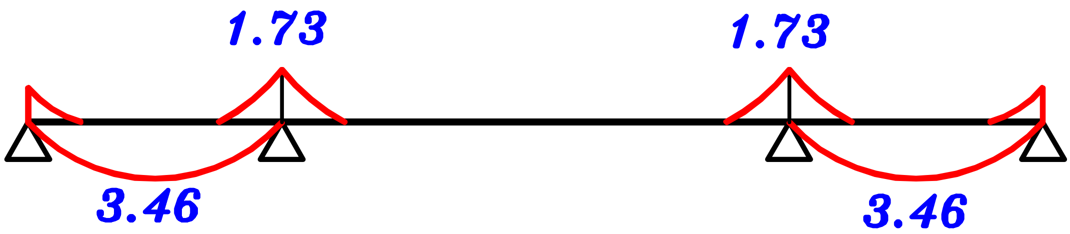
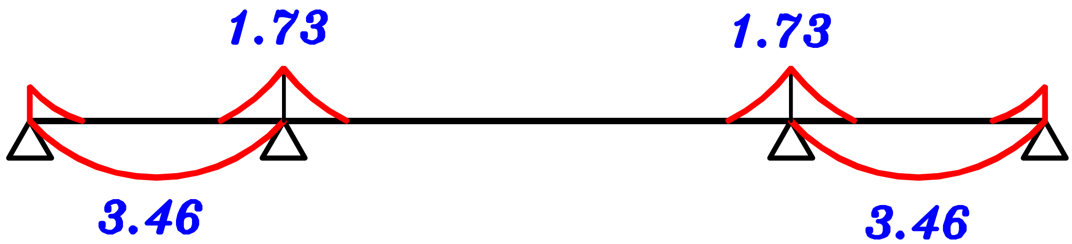
٥ – نعمل تصميم للشرائح مع مراعاه عرض الشريحه .

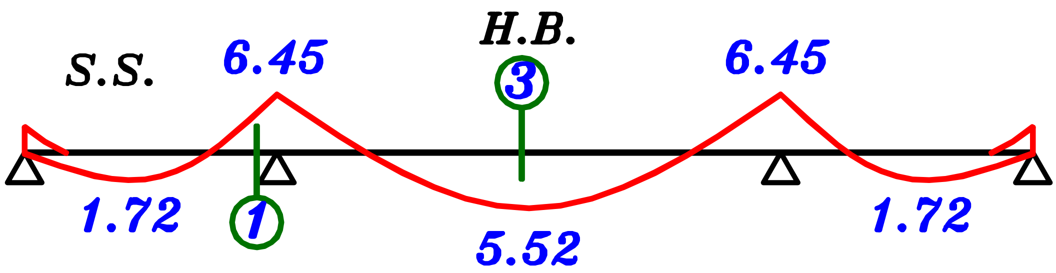
$S = 1.0\,m$



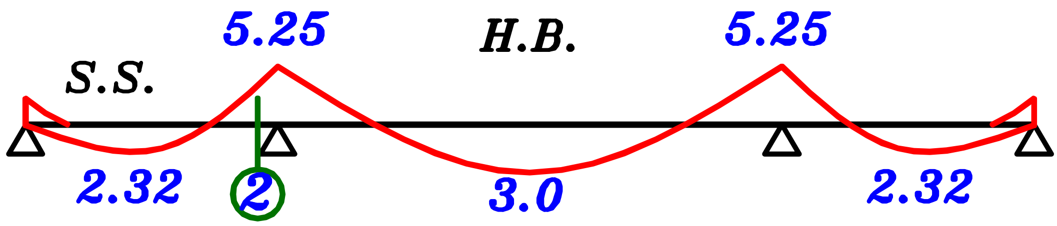
$S = 1.0\,m$



$S = 0.5\,m$



$S = 0.5\,m$



Sec. 1 S.S. $M_{U.L.} = 6.45$ kN.m\rib

$t = 100\,mm$, $d = 100 - 20 = 80\,mm$, $S = 500\,mm$ عرض الشريحه

$$80 = C_1\sqrt{\frac{6.45 * 10^6}{25 * 500}} \longrightarrow C_1 = 3.52 \longrightarrow J = 0.780$$

$$A_s = \frac{6.45 * 10^6}{0.780 * 360 * 80} = 287.1\ mm^2/0.5\,m$$

$$A_s = \frac{287.1}{0.50} = 574.2\ mm^2/m'$$

$\boxed{8\phi 10\backslash m'}$ عدد زوجي

Sec. ② S.S. $M_{U.L.} = 5.25$ kN.m\rib

$t = 100\ mm$, $d = 100 - 20 = 80\ mm$, $S = 500\ mm$ عرض الشريحه

$$80 = C_1\sqrt{\frac{5.25 * 10^6}{25 * 500}} \longrightarrow C_1 = 3.90 \longrightarrow J = 0.80$$

$$A_s = \frac{5.25 * 10^6}{0.80 * 360 * 80} = 227.8\ mm^2/0.5\ m$$

$$A_s = \frac{227.8}{0.50} = 455.7\ mm^2/m`$$

$6\phi10\backslash m`$ عدد زوجى

Sec. ③ H.B. $M_{U.L.} = 5.52$ kN.m\rib

$t = 200\ mm$, $d = 200 - 30 = 170\ mm$, $S = 500\ mm$ عرض الشريحه

$$170 = C_1\sqrt{\frac{5.52 * 10^6}{25 * 500}} \longrightarrow C_1 = 8.09 \longrightarrow J = 0.826$$

$$A_s = \frac{5.52 * 10^6}{0.826 * 360 * 170} = 109.1\ mm^2/rib$$

$2\phi10\backslash rib$

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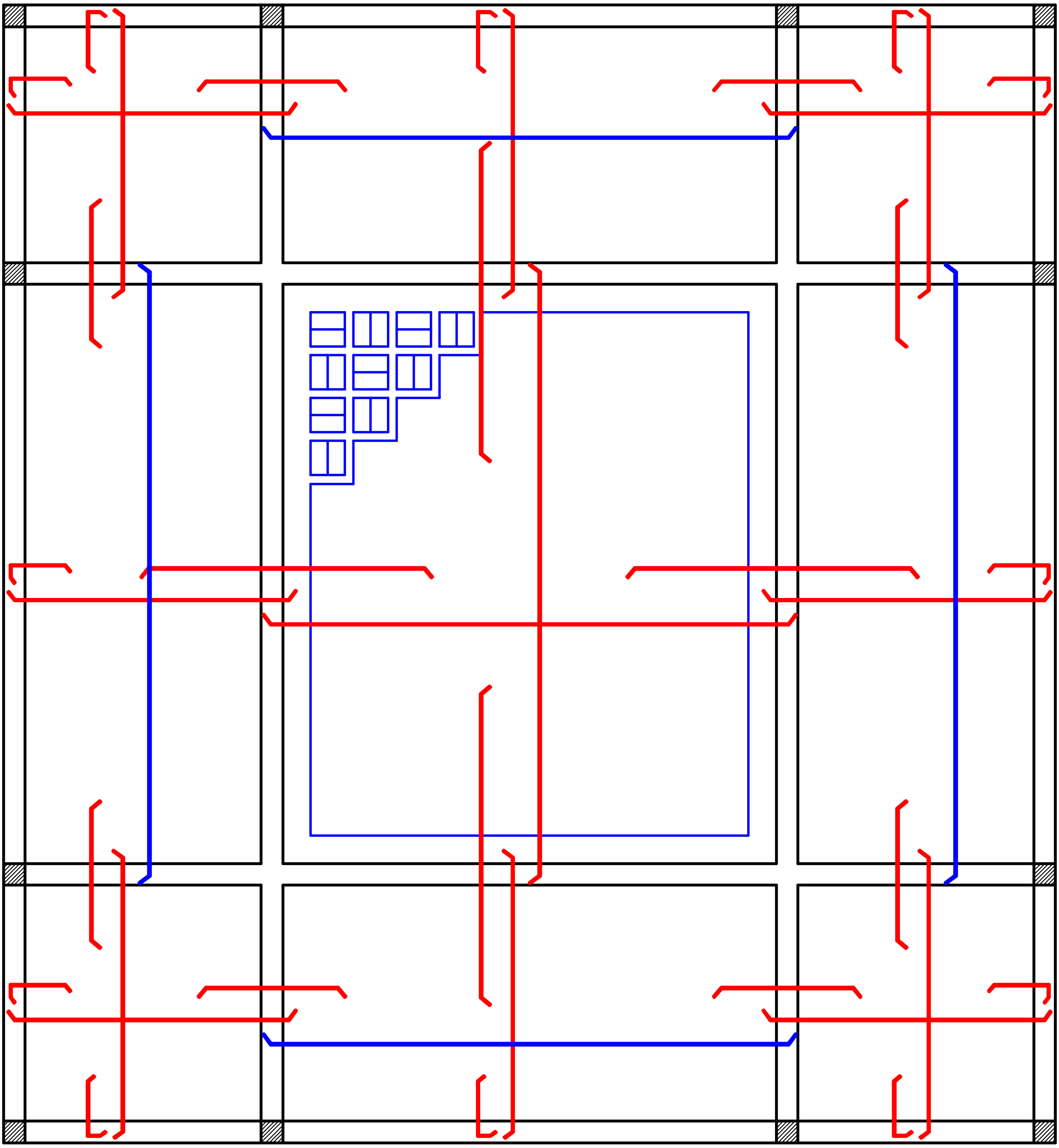
RFT. of the slabs.



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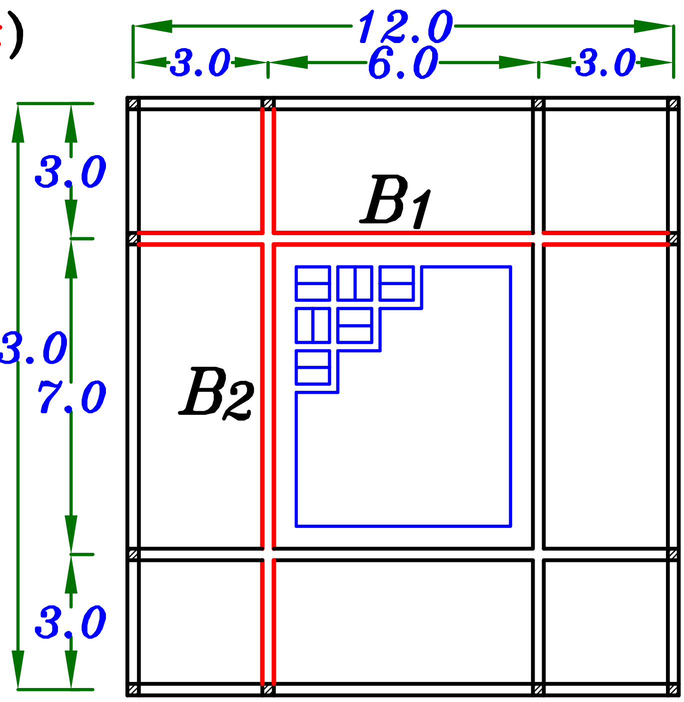
Design of Panelled Beams.

a – Get the Dimensions of the beam. (b, t)



Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{12.0}{16} = 0.75\ m$$

$$\boxed{b = 0.25\ m} \quad \boxed{t = 0.75\ m}$$

b – Get the Loads on the Slab. (w_{av})

$$w_{av.} = \frac{\text{Total Weight of Solid slabs} + \text{Total Weight of H.B. slabs} + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

$$\boxed{w_{av.} = \frac{w_s * area\,(s.s.) + \left(\frac{w_{rib}}{S}\right) * area\,(H.B.) + \text{Total Weight of Panelled Beams}}{\text{Total area}}}$$

$$w_{av.} = \frac{w_s * (L * L_s - a * b) + \left(\frac{w_{rib}}{S}\right) * (a * b) + \left[1.4 * b\,(t - t_s)\,(2L + 2L_s) * \gamma_c\right]}{L * L_s}$$

$$w_{av.} = \frac{8.80\,(12*13 - 6*7) + \left(\frac{5.03}{0.5}\right)(6*7) + 1.4 * 0.25\,(0.75 - 0.1)\,[2*12 + 2*13] * 25}{12*13}$$

$$= 10.96\ kN\backslash m^2$$

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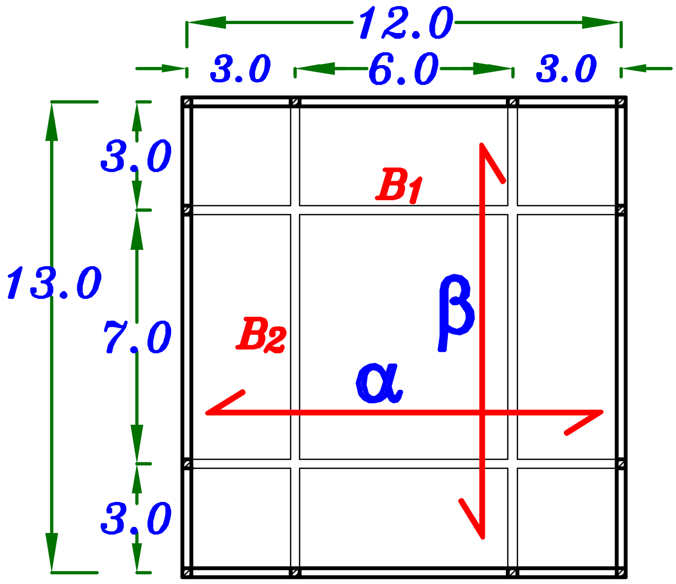
C – Calculate α , β By using Grashoff.

$$r = \frac{m\,L}{m'\,L_s} = \frac{(1.0)\,13.0}{(1.0)\,12.0} = 1.083$$

$$\alpha = \frac{r^4}{1+r^4} = \frac{(1.083)^4}{1+(1.083)^4} = 0.58$$

$$\beta = \frac{1}{1+r^4} = \frac{1}{1+(1.083)^4} = 0.42$$



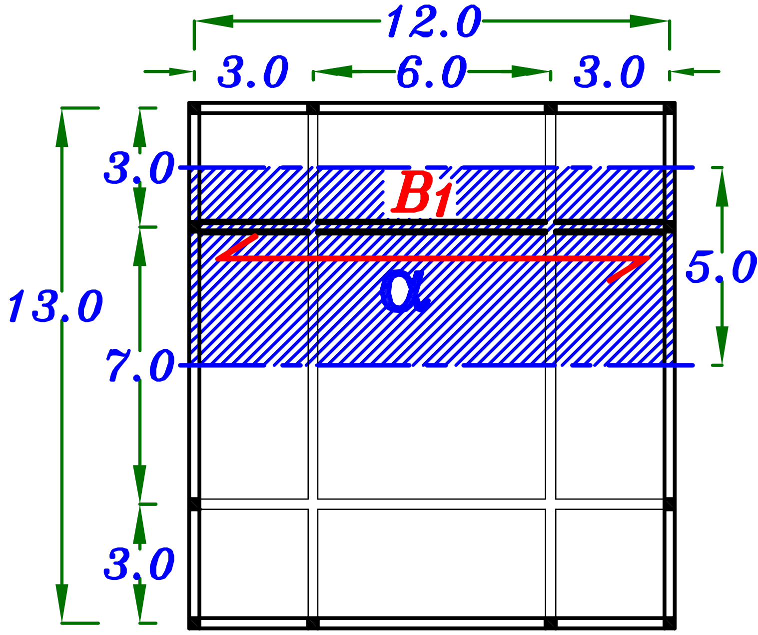
B_1 α Direction.

$a = 5.0\ m$

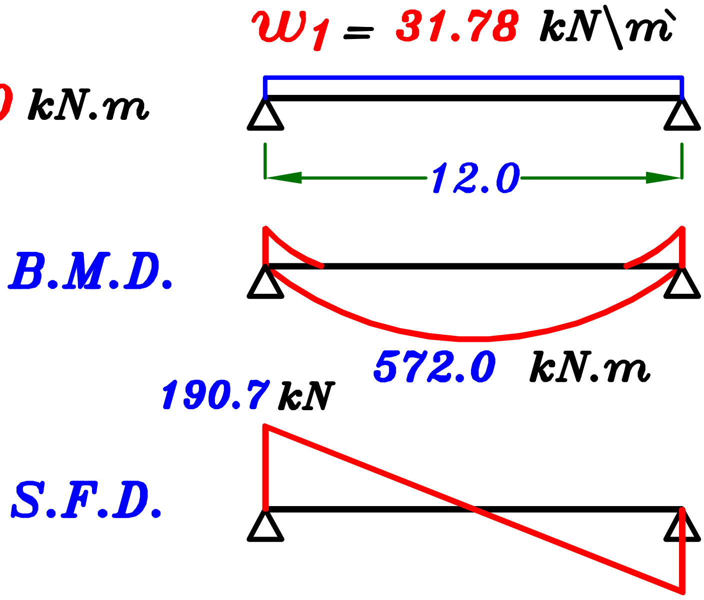
$$w_1 = w_{av.} * a * \alpha$$

$$= 10.96 * 5.0 * 0.58$$

$$= 31.78\ kN\backslash m'$$



$$M = 31.78 * \frac{12.0^2}{8} = 572.0\ kN.m$$



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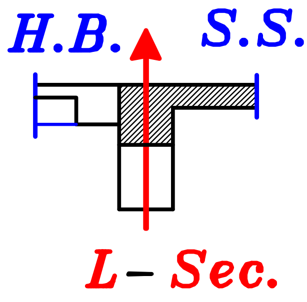
e- Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 3.0\ m\ ,\ \frac{L}{2} = 6.5\ m \qquad \theta_{B_1} = \frac{3.0}{6.5} * 90^\circ = 41.52^\circ$$

$$M_1 = 572.0 * \frac{\sin 41.52^\circ}{\sin 90^\circ} = 379.17\ kN.m$$

F- Design the Panelled Beam. B_1



α Direction. $\therefore$ Cover = 50 mm

$$t = 750\ mm \qquad d = 750-50 = 700\ mm$$

$$B = \begin{cases} C.L.-C.L. = 1.5\ m = 1500\ mm \\ 6\,t_s + b = 6 * 100 + 250 = 850\ mm \\ K\frac{L}{10} + b = 1.0 * \frac{12000}{10} + 250 = 1450\ mm \end{cases} \qquad \boxed{B = 850\ mm}$$

$$700 = C_1 \sqrt{\frac{379.17 * 10^6}{25 * 850}} \longrightarrow C_1 = 5.24 \longrightarrow J = 0.826$$

$$A_s = \frac{379.17 * 10^6}{0.826 * 360 * 700} = 1821.6\ mm^2$$

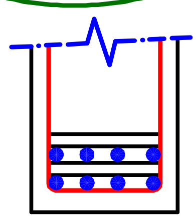
Check $A_{s_{min.}}$ $\qquad A_{s_{req.}} = 1821.6\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 700 = 546.87\ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1821.6\ mm^2 \qquad 8\phi18$$

Stirrup Hangers = $(0.1 \rightarrow 0.2)\ A_s = (182 \rightarrow 364\ mm^2)$

$3\phi10$



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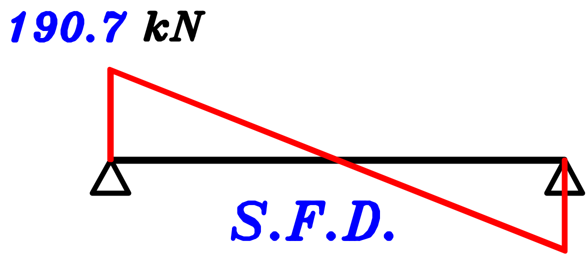
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$



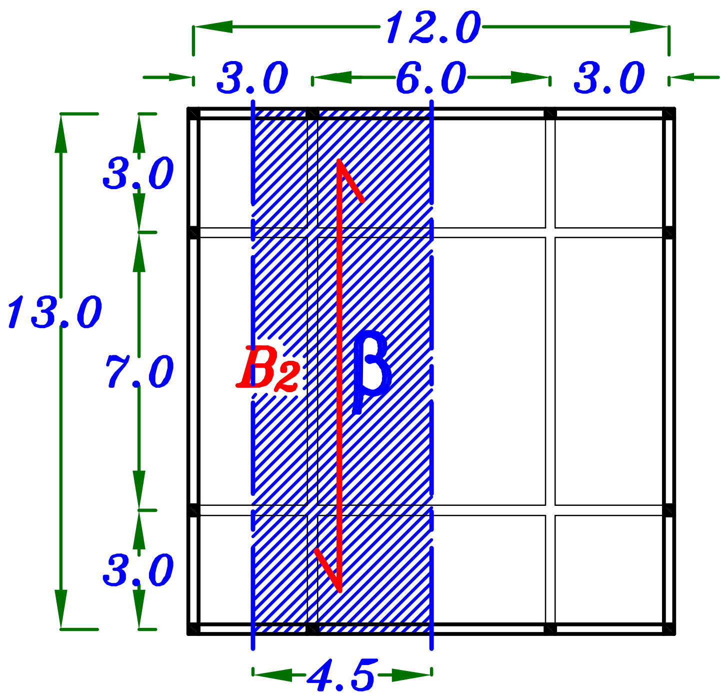
$$q_s = \frac{Q_{max}}{b\,d} = \frac{190.7 * 10^3}{250 * 700} = 1.09 N\backslash mm^2 \qquad \therefore q_{cu} < q_s < q_{u_{max}}$$

$$q_s - \frac{q_{cu}}{2} = \frac{n A_s (F_y \backslash \gamma_s)}{b\ S} \xrightarrow{Take} n = 2 \ , \ \phi 8 = 50.3 \ mm^2$$

$$1.09 - \frac{0.98}{2} = \frac{2\,(50.3)\,(240\backslash 1.15)}{250 * S} \longrightarrow S = 139.9 \ mm$$

No. of stirrups\m` $= \frac{1000}{S} = \frac{1000}{139.9} = 7.14$ Use $8\phi 8\backslash m$`

$\underline{B_2}$ β Direction.



$$b = 4.5\, m$$

$$w_2 = w_{av.} * b * \beta$$

$$= 10.96 * 4.5 * 0.42$$

$$= 20.71 \ kN\backslash m`$$

$$M = 20.71 * \frac{13.0^2}{8} = 437.5 \ kN.m$$

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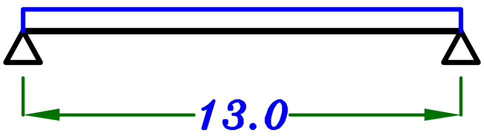
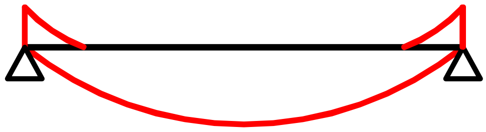
$w_2 = 20.71 \ kN\backslash m'$



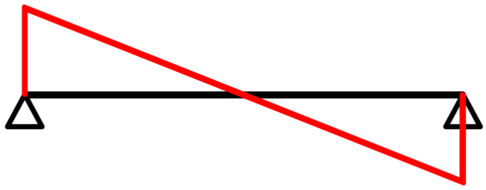
B.M.D.



437.5 kN.m

134.6 kN

S.F.D.



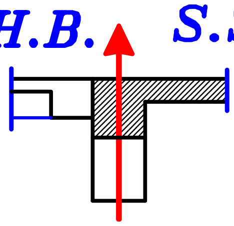
e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin \theta}{\sin 90}\right)$

$$X = 3.0 \ m \ , \ \frac{L}{2} = 6.0 \ m \qquad \theta_{B_1} = \frac{3.0}{6.0} * 90^\circ = 45^\circ$$

$$M_1 = 437.5 * \frac{\sin 45^\circ}{\sin 90^\circ} = 309.3 \ kN.m$$

F – Design the Panelled Beam. B_1

H.B. S.S.



L – Sec.

α Direction. $\therefore$ Cover = 70 mm

$$t = 750 \ mm \qquad d = 750 - 70 = 680 \ mm$$

$$B = \left\{ \begin{array}{l} C.L.-C.L. = 1.5 \ m = 1500 \ mm \\ 6 \ t_s + b = 6 * 100 + 250 = 850 \ mm \\ K \frac{L}{10} + b = 1.0 * \frac{13000}{10} + 250 = 1550 \ mm \end{array} \right\} \quad \boxed{B = 850 \ mm}$$

$$680 = C_1 \sqrt{\frac{309.3 * 10^6}{25 * 850}} \longrightarrow C_1 = 5.63 \longrightarrow J = 0.826$$

$$A_s = \frac{309.3 * 10^6}{0.826 * 360 * 680} = 1529.6 \ mm^2$$

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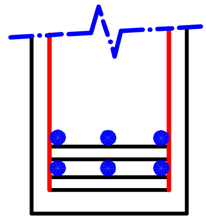
Check $A_{s_{min.}}$ $A_{s_{req.}} = 1529.6 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 680 = 531.25 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1529.6 \ mm^2$ $6\phi18$

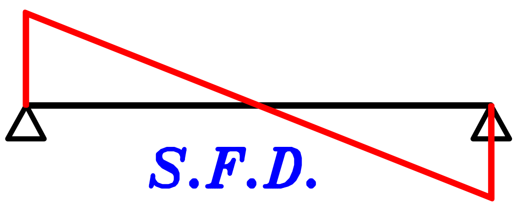


Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (148 \rightarrow 296 \ mm^2)$ $2\phi10$

Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$



$$q_s = \frac{Q_{max}}{b\, d} = \frac{134.6 * 10^3}{250 * 700} = 0.77 \ N\backslash mm^2$$

$\therefore q_s < q_{cu}$

$\therefore$ Use min. Shear RFT. $5\phi8\backslash m'$

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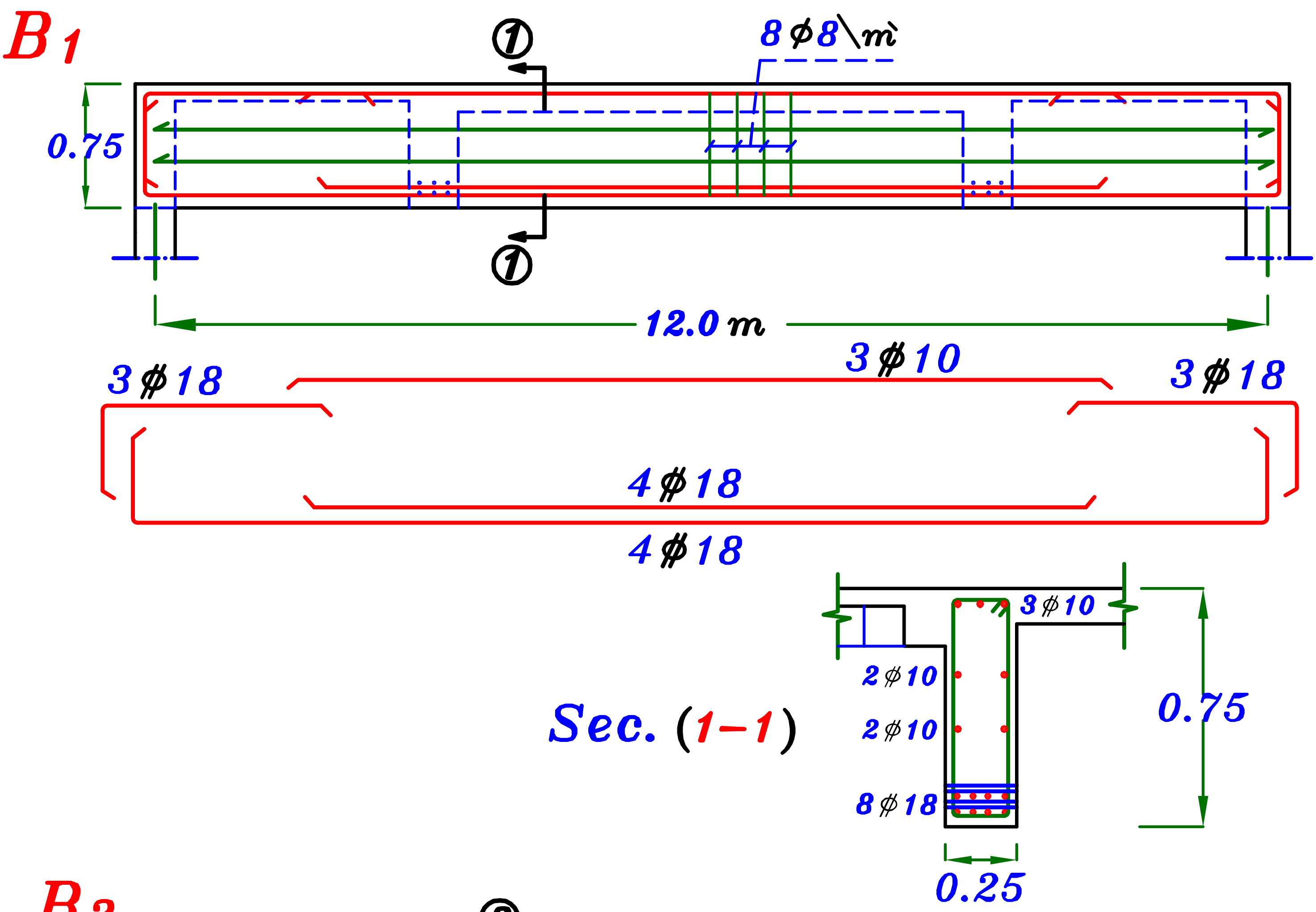
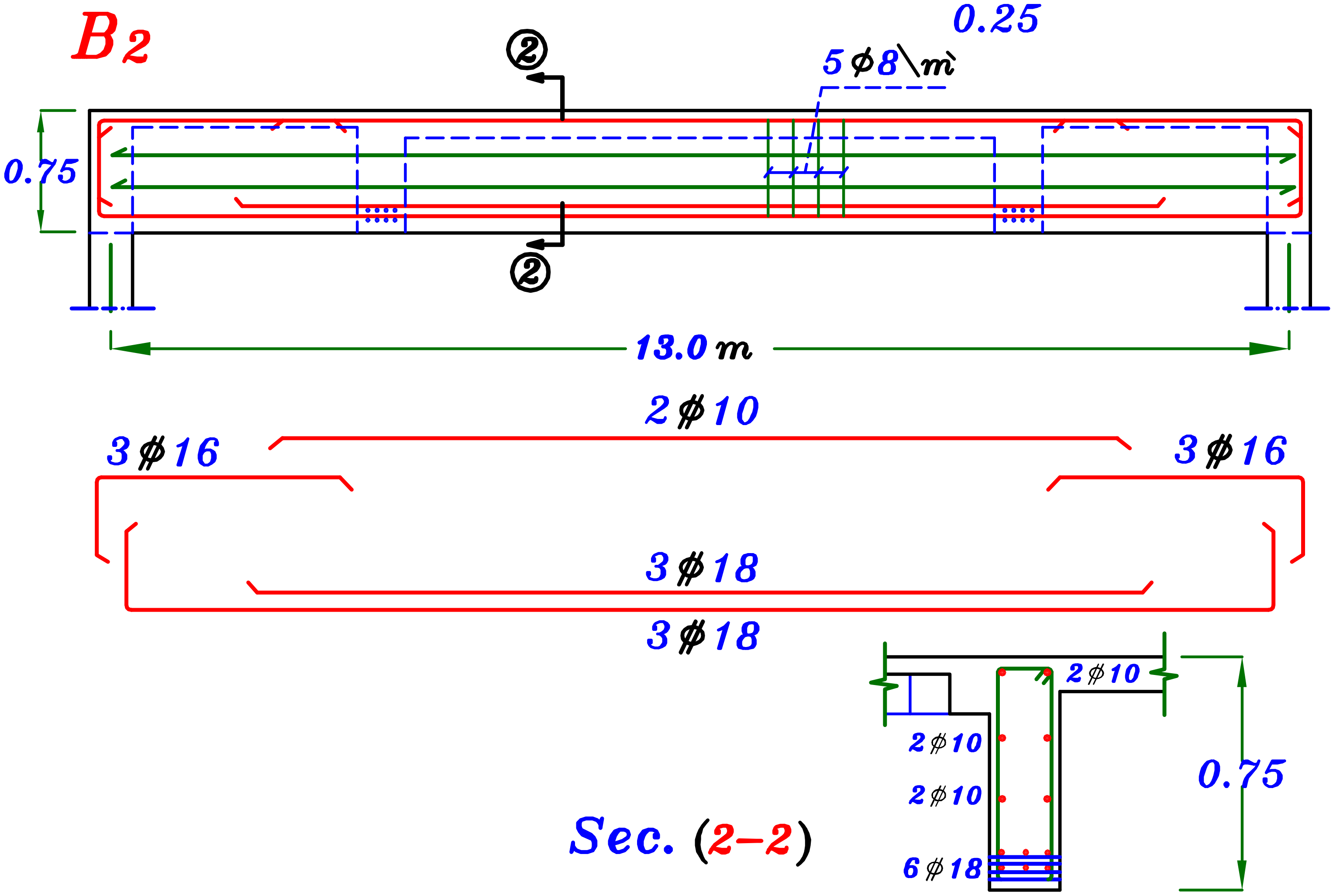
Draw Details of RFT. For the Beams.(B_1, B_2)





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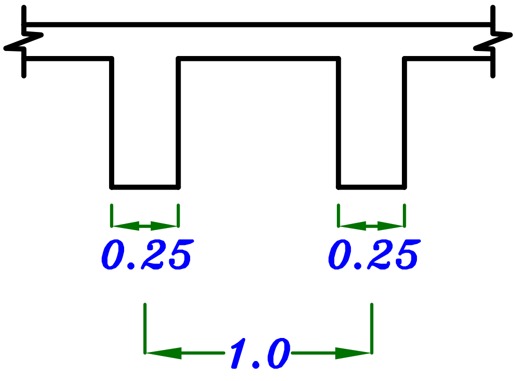
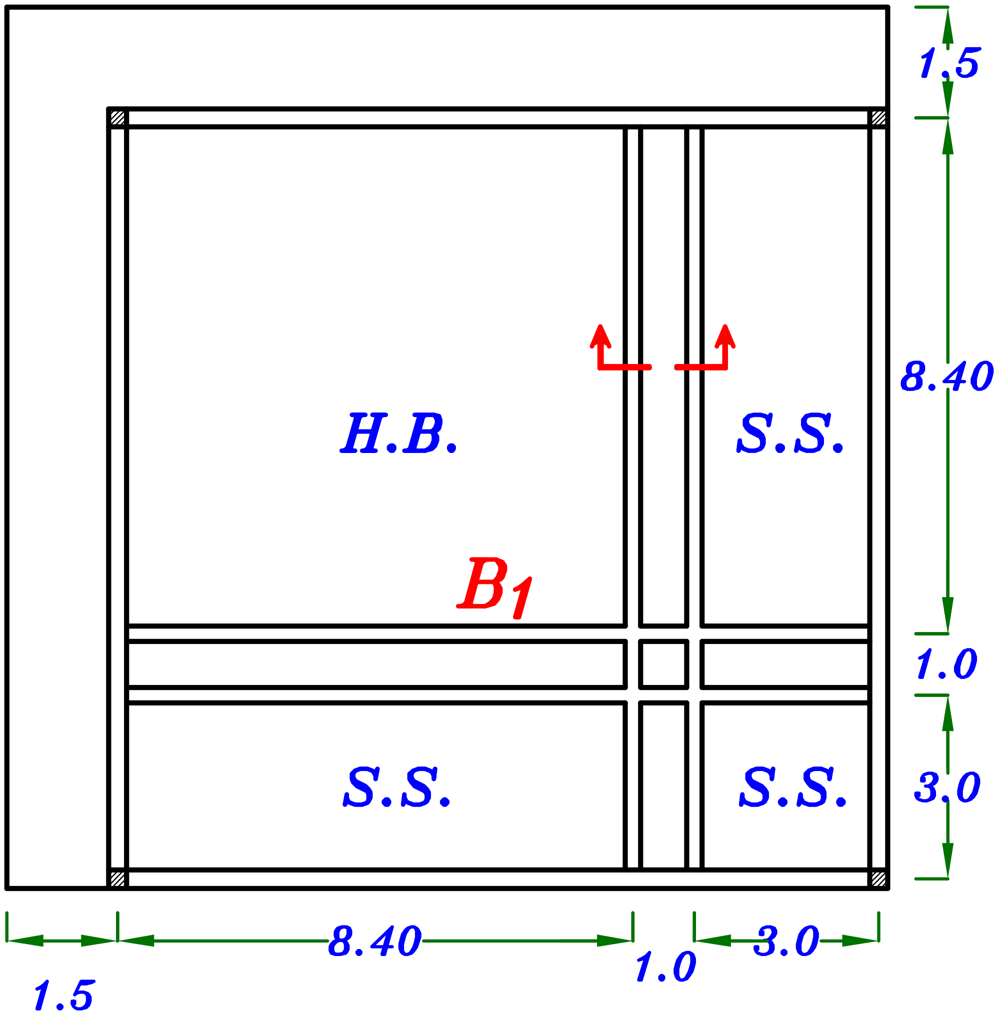
Example.





Data.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. = 2.0\ kN\backslash m^2$, $L.L. = 2.0\ kN\backslash m^2$

Req.

1. Design the slab & Draw Details of RFT.
2. Design the panelled Beam B_1 & Draw Details of RFT. in elevation.

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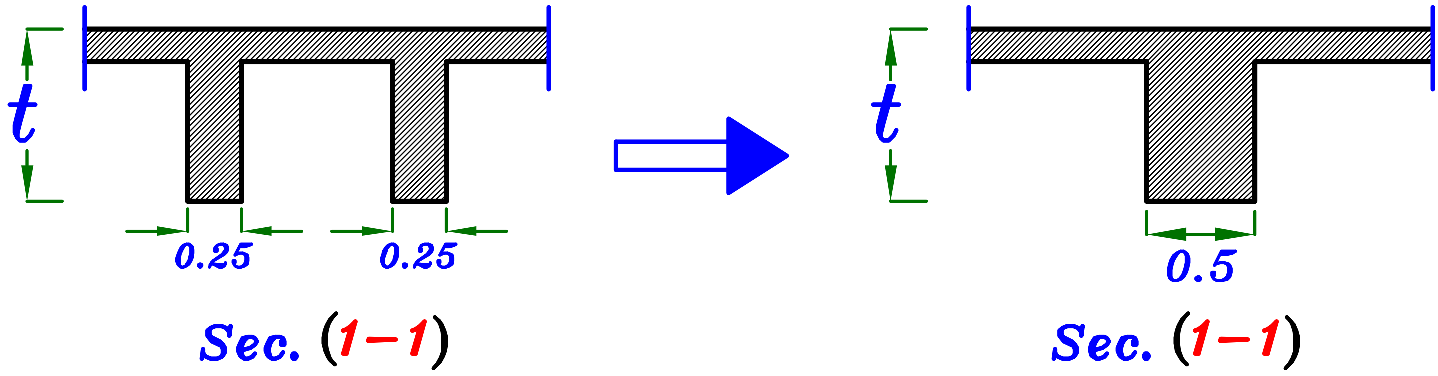
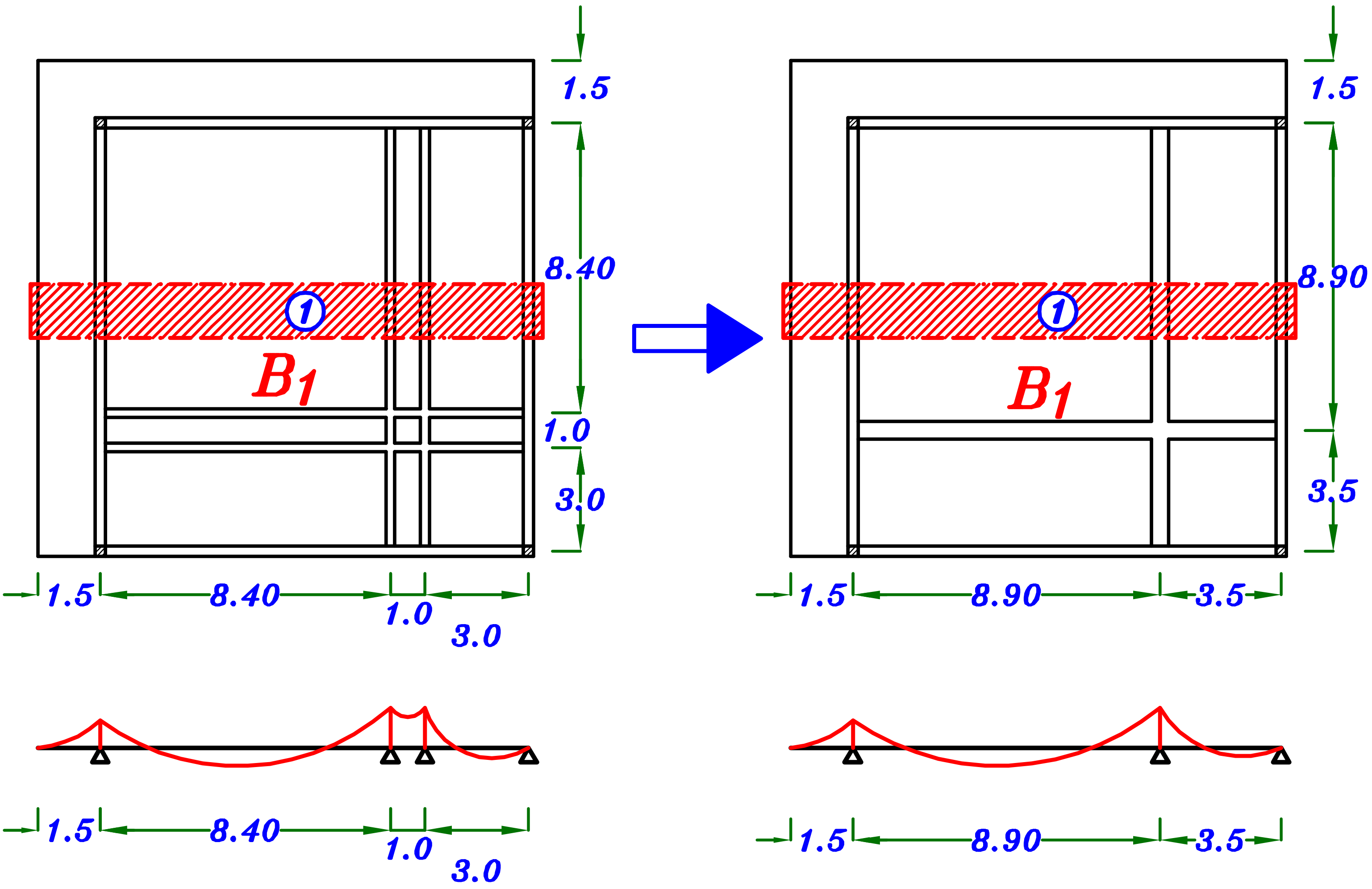
فكره المسأله .

ممكن للتسهيل عند حساب الاحمال و تصميم كلا من البلاطه و الكمره اعتبار انه تم ضم الكمرتين المتجاورتين معا و اعتبارهم فى الحسابات كأنها كمره واحده .



و ذلك أثناء حسابات الاحمال و التصميم فقط لكن عند رسم التسليح يجب ان نسلح على الشكل الاصلى . لان هذا ما سيتم تنفيذه فى الحقيقه



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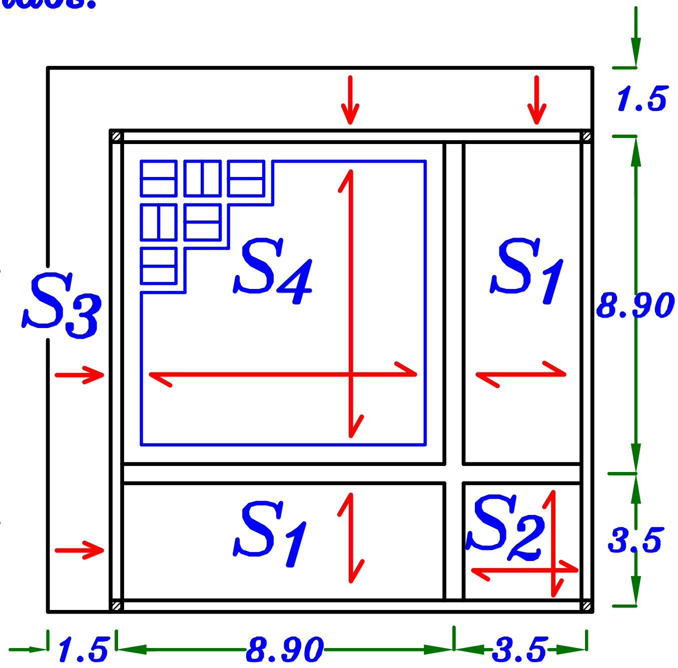
a- Choose the Thickness of the Slabs.



a - Choose the Thickness of the Slab. (t_s)

S_1 One way $L_s = 3.5\ m$

$$t_s = \frac{3500}{30} = 116.6\ mm$$

S_2 two way $L_s = 3.5\ m$

$$t_s = \frac{3500}{40} = 87.5\ mm$$

S_3 Cantilever $L_c = 1.5\ m$

$$t_s = \frac{1500}{10} = 150\ mm$$

Take (t_s) the bigger value $\boxed{t_s = 150\ mm}$

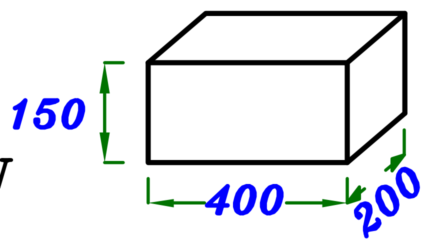
S_4 Two way H.B. $L_s = 8.90\ m$

$$t = \frac{8900}{45} = 197\ mm \qquad \boxed{t = 200\ mm}$$

Take $\boxed{t = 200\ mm}$ $\boxed{t_s = 50\ mm}$ $\boxed{h = 150\ mm}$

The Block $(200 * 400 * 150)$

$h = 150\ mm \longrightarrow$ Weight of Block $= 100\ N$



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b- Get the Loads on the Slab

For Solid Slabs.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\ L.L.$$

$$w_s = 1.4\,(0.15 * 25 + 2.0) + 1.6\,(2.0) = 11.25\ kN\backslash m^2$$

For Two way Hollow Blocks.

$$\boxed{\begin{aligned} w_{ribT} &= [1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)]\,(S*S) \\ &+1.4*b\,h*(2S-b)*\gamma_c + 1.4*(\text{Block وزن ال})\left(\frac{e}{a}\right)\end{aligned}}$$

$$\therefore\ w_{ribT} = [1.4\,(0.05*25 + 2.0) + 1.6\,(2.0)]\,(0.5*0.5)$$

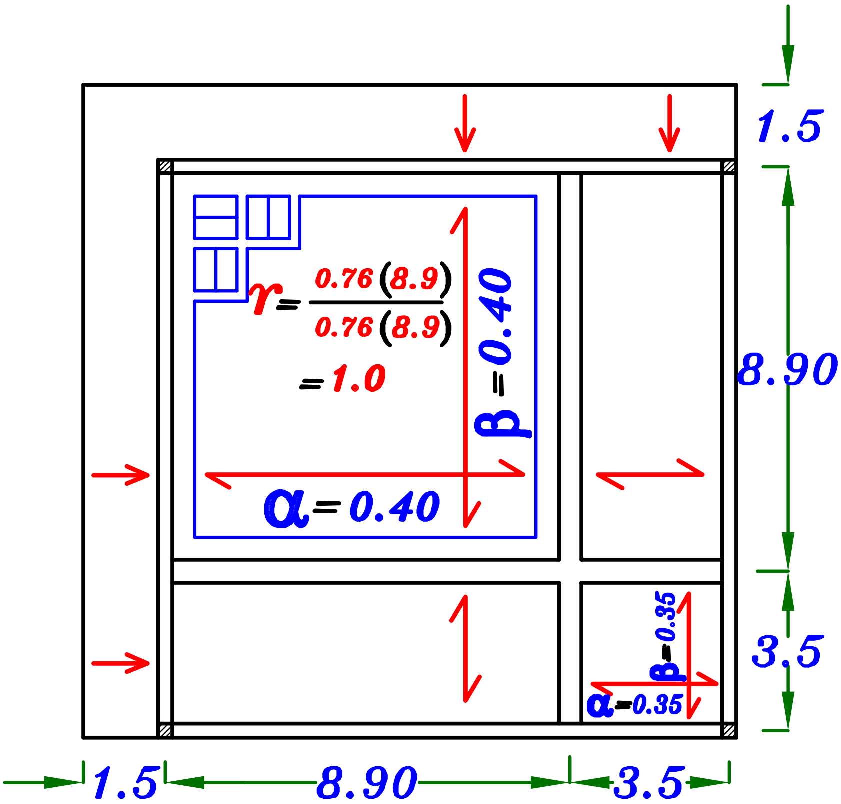
$$+1.4\,(0.1*0.15*(2*0.5-0.1)*25) + 1.4\left(\frac{100}{1000}\right)\left(\frac{0.4}{0.2}\right) = 2.69$$

$$(kN\backslash(S*2S))$$

$$w_{rib} = \frac{w_{ribT}}{S} = \frac{2.69}{0.5} = 5.38\ kN\backslash(S*m)$$

$$\boxed{\begin{array}{l}\text{For H.B.} \\ \text{Use Marcus}\end{array}}$$

$$\boxed{\begin{array}{l}\text{For S.S.} \\ \text{Use C.P.}\end{array}}$$



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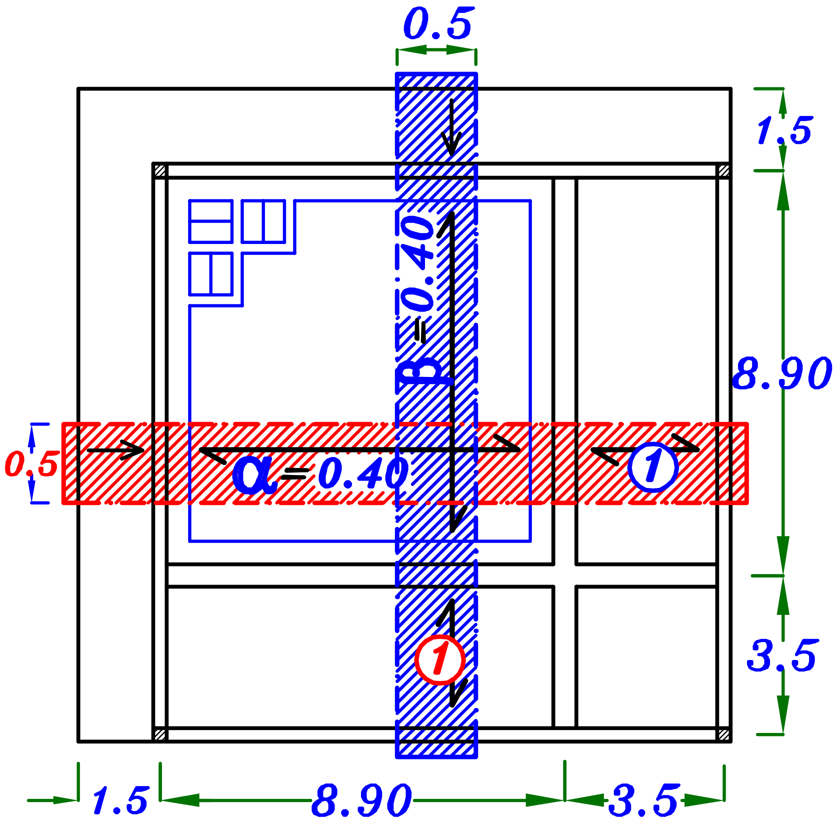
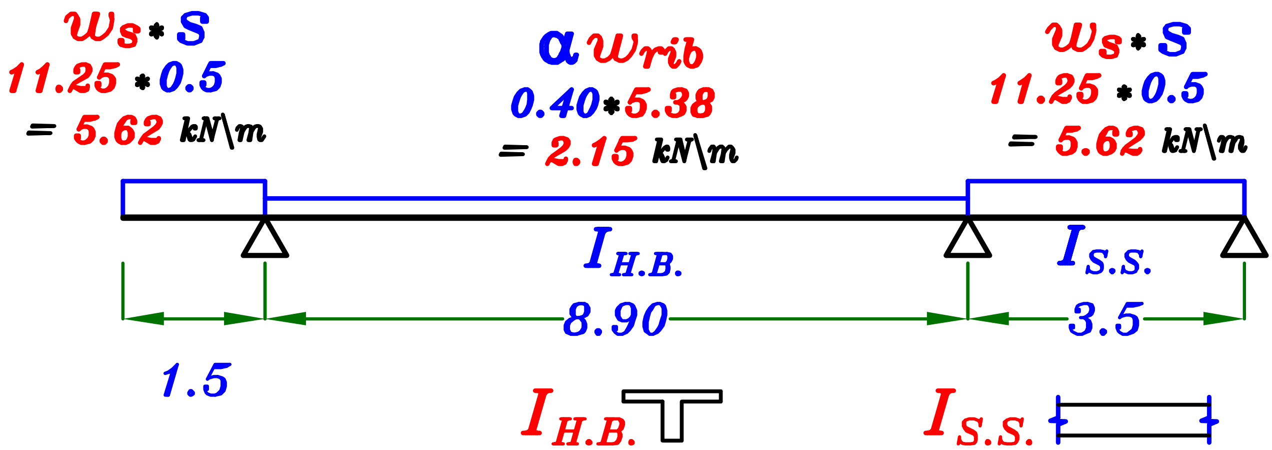
Strip ①

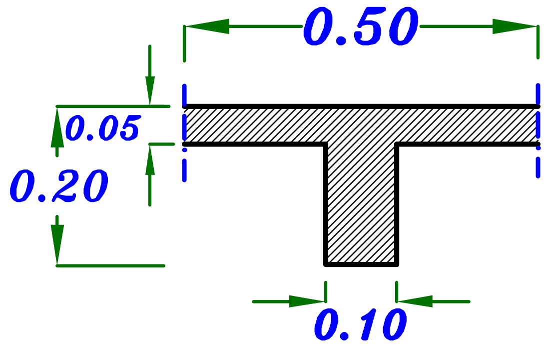




$I_{H.B.} =$ (T-section) $\qquad I_1 = (\mu * 10^{-4})\, B\, t^3$

$B = 0.5\ m \quad , \quad t = 0.20\, m$

$\dfrac{t_s}{t} = \dfrac{0.05}{0.20} = 0.25 \;, \quad \dfrac{b_o}{B} = \dfrac{0.1}{0.5} = 0.2$



From Tables page 91 $\quad \mu = 318$

$I_{H.B.} = (318 * 10^{-4} * 0.5 * 0.20^3) = 1.27 * 10^{-4}\ m^4$

$I_{S.S.} = \dfrac{S\,(t_s)^3}{12} = \dfrac{0.5\,(0.15)^3}{12} = 1.40 * 10^{-4}\ m^4$

$\therefore \dfrac{I_{H.B.}}{I_{S.S.}} = \dfrac{1.27 * 10^{-4}}{1.40 * 10^{-4}} = 0.907 \qquad \therefore \boxed{I_{H.B.} = 0.907\, I_{S.S.}}$

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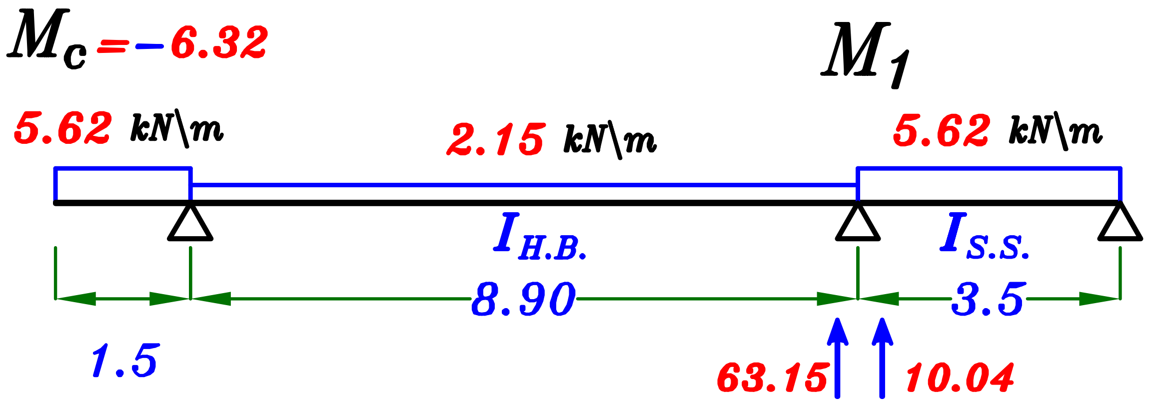
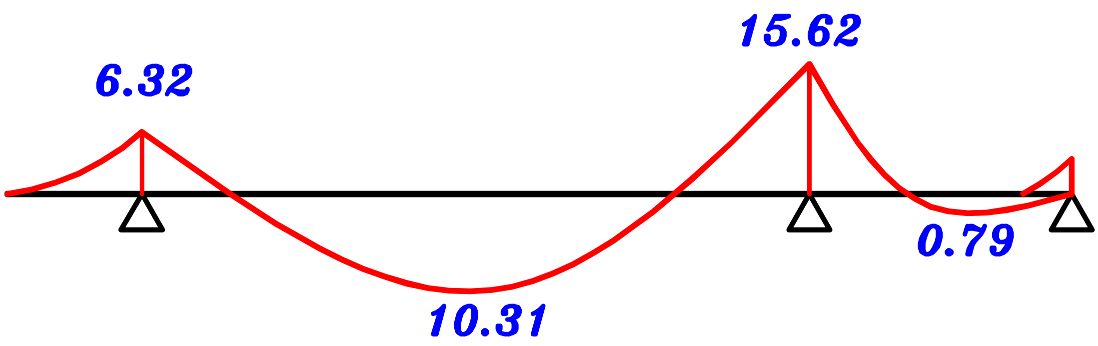
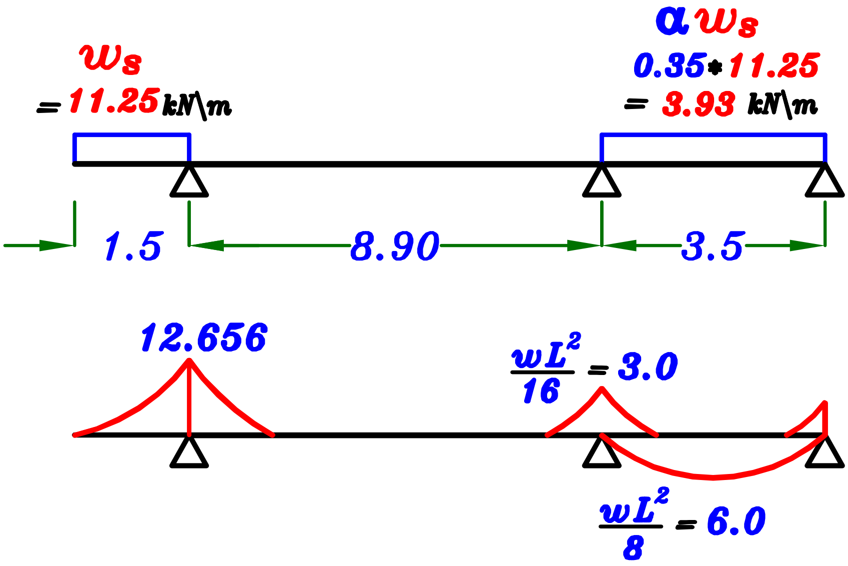


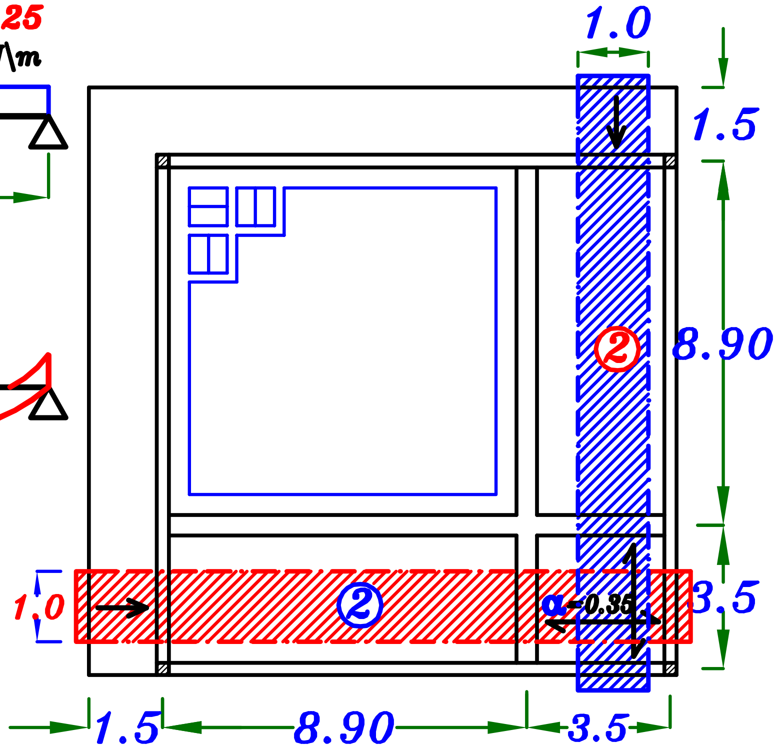
$$-6.32\left(\frac{8.90}{0.907\, I_{s.s.}}\right) + 2M_1\left(\frac{8.90}{0.907\, I_{s.s.}} + \frac{3.5}{I_{s.s.}}\right) + 0.0 = -6\left(\frac{63.15}{0.907\, I_{s.s.}} + \frac{10.04}{I_{s.s.}}\right)$$

$$\boxed{M_1 = -15.45 \text{ kN.m}\backslash 0.5\text{ m}}$$



Strip ②





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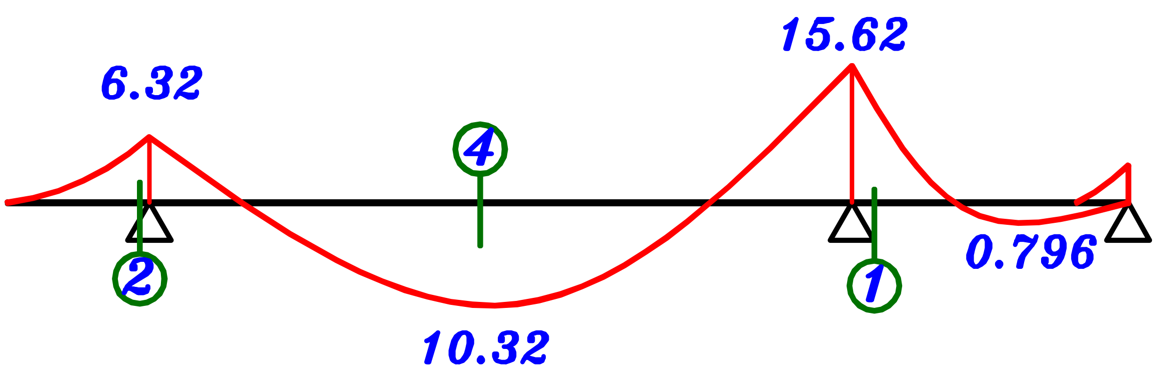

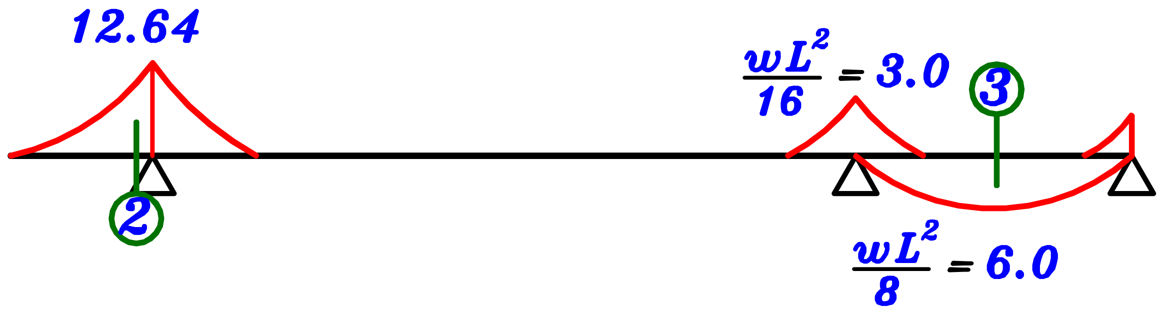

Sec. ① S.S. $M_{U.L.} = 15.62$ kN.m\rib

$t = 150$ mm , $d = 150 - 20 = 130$ mm , $S = 500$ mm عرض الشريحة

$$130 = C_1 \sqrt{\frac{15.62 * 10^6}{25 * 500}} \longrightarrow C_1 = 3.678 \longrightarrow J = 0.791$$

$$A_s = \frac{15.62 * 10^6}{0.791 * 360 * 130} = 421.95 \, mm^2/0.5\, m$$

$$A_s = \frac{421.95}{0.50} = 843.9 \, mm^2/m'$$ $\boxed{8 \phi 12 \backslash m'}$ عدد زوجى

Sec. ② S.S. $M_{U.L.} = 6.32$ kN.m\rib

$t = 150$ mm , $d = 150 - 20 = 130$ mm , $S = 500$ mm عرض الشريحة

$$130 = C_1 \sqrt{\frac{6.32 * 10^6}{25 * 500}} \longrightarrow C_1 = 5.78 \longrightarrow J = 0.826$$

$$A_s = \frac{6.32 * 10^6}{0.826 * 360 * 130} = 163.5 \, mm^2/0.5\, m$$

$$A_s = \frac{163.5}{0.50} = 326.98 \, mm^2/m'$$ $\boxed{6 \phi 10 \backslash m'}$ عدد زوجى

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Sec. ③ S.S. $M_{U.L.} = 6.0$ kN.m\rib

$t = 150\ mm$ ، $d = 150 - 20 = 130\ mm$ ، $S = 1000\ mm$ عرض الشريحه

$$130 = C_1 \sqrt{\frac{6.0 * 10^6}{25 * 1000}} \longrightarrow C_1 = 8.39 \longrightarrow J = 0.826$$

$$A_s = \frac{6.0 * 10^6}{0.826 * 360 * 130} = 155.2\ mm^2/m$$

$5 \phi 10 \backslash m$

Sec. ④ H.B. $M_{U.L.} = 10.32$ kN.m\rib

$t = 200\ mm$ ، $d = 200 - 30 = 170\ mm$ ، $S = 500\ mm$ عرض الشريحه

$$170 = C_1 \sqrt{\frac{10.32 * 10^6}{25 * 500}} \longrightarrow C_1 = 5.917 \longrightarrow J = 0.826$$

$$A_s = \frac{10.32 * 10^6}{0.826 * 360 * 170} = 204.11\ mm^2/rib$$

$2 \phi 12 \backslash rib$

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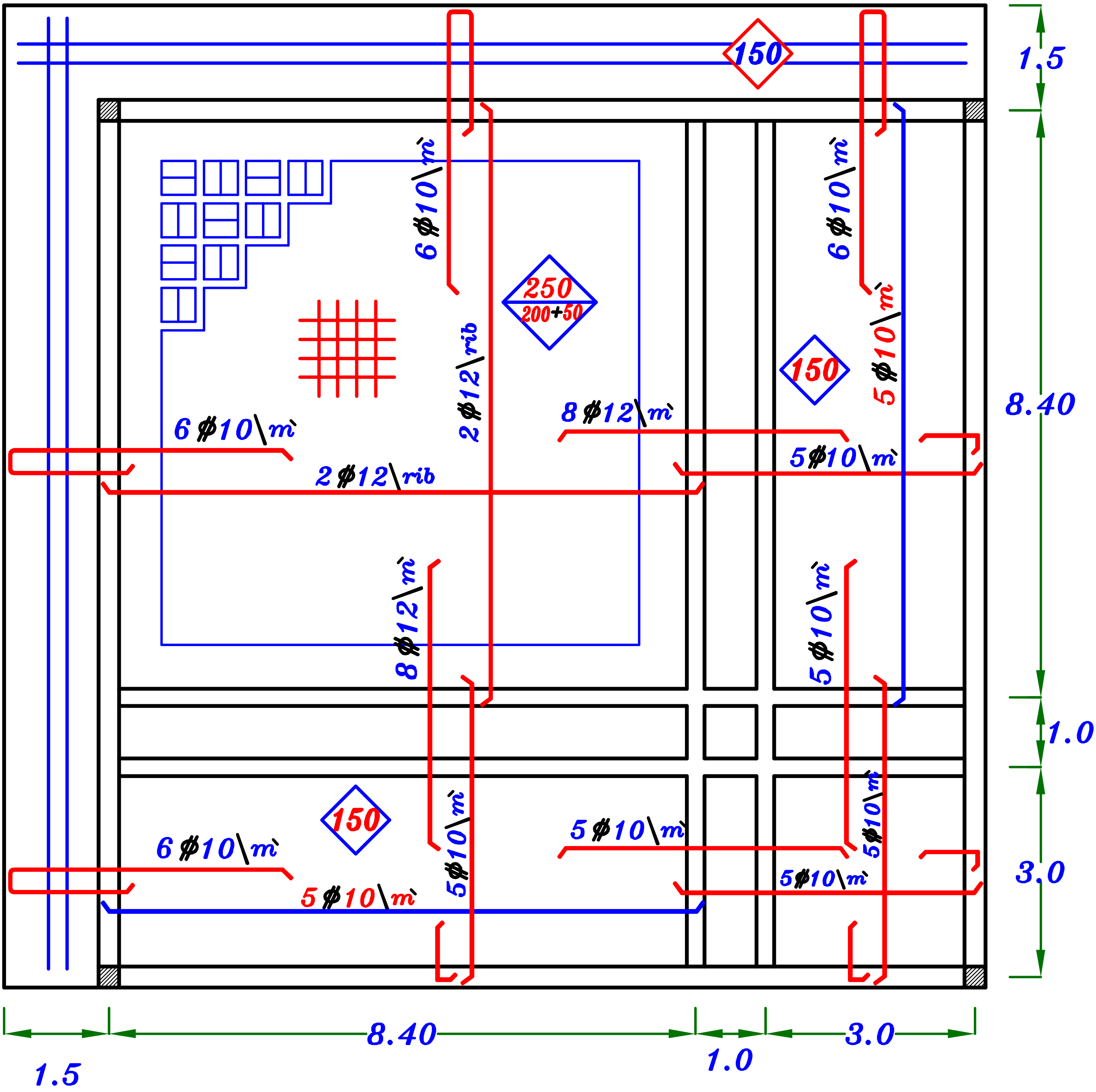
RFT. of the slabs.



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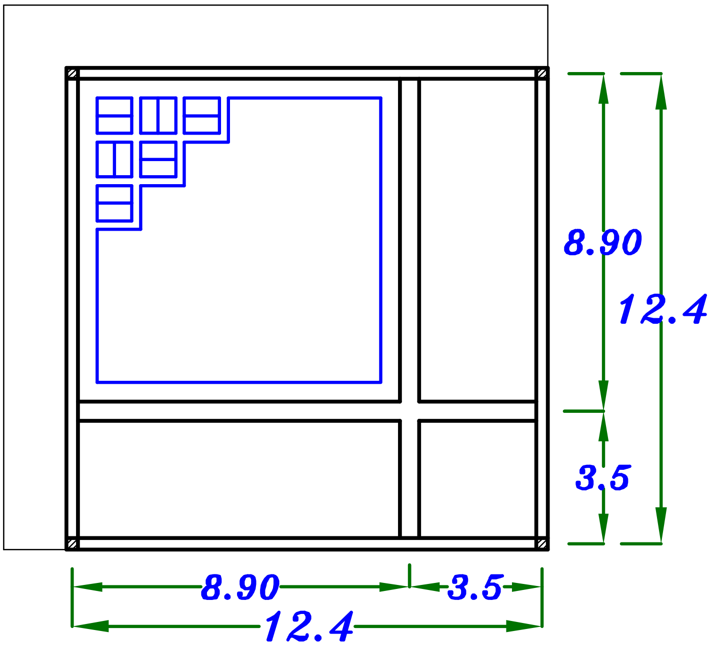
Design of Panelled Beams.

a – Get the Dimensions of the beam. (b, t)

Take $b = 0.50\ m$

$$t = \frac{L_s}{16} = \frac{12.4}{16} = 0.775\ m$$

$$= 0.80\ m$$



$$\boxed{b = 0.25\ m} \quad \boxed{t = 0.80\ m}$$

b – Get the Loads on the Slab. (w_{av})

$$w_{av.} = \frac{\text{Total Weight of Solid slabs} + \text{Total Weight of H.B. slabs} + \text{Total Weight of Panelled Beams}}{\text{Total area}}$$

$$\boxed{w_{av.} = \frac{w_s \cdot area\,(s.s.) + \left(\frac{w_{rib}}{S}\right) \cdot area\,(H.B.) + \text{Total Weight of Panelled Beams}}{\text{Total area}}}$$

$$w_{av.} = \frac{w_s * (L * L_s - a * b) + \left(\frac{w_{rib}}{S}\right) * (a * b) + \left[1.4 * b\,(t - t_s)\,(2L) * \gamma_c\right]}{L * L_s}$$

$$w_{av.} = \frac{11.25\,(12.4 * 12.4 - 8.9 * 8.9) + \left(\frac{5.38}{0.5}\right)(8.9 * 8.9) + 1.4 * 0.50\,(0.80 - 0.15)\,[2 * 12.4] * 25}{12.4 * 12.4}$$

$$= 12.83\ kN\backslash m^2$$

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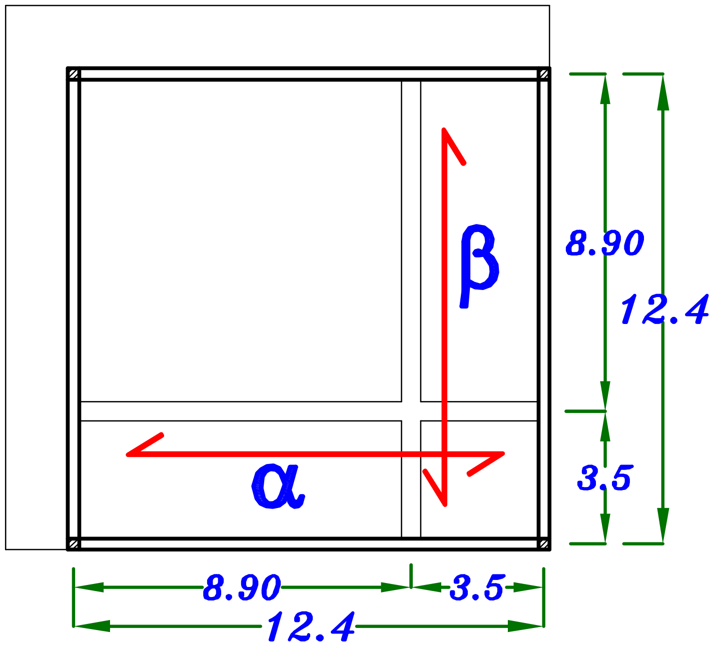
C – Calculate α , β By using Grashoff.

$$r = \frac{m\, L}{m'\, L_s} = \frac{(1.0)\ 12.4}{(1.0)\ 12.4} = 1.0$$

$$\alpha = \frac{r^4}{1+r^4} = \frac{(1.0)^4}{1+(1.0)^4} = 0.50$$

$$\beta = \frac{1}{1+r^4} = \frac{1}{1+(1.0)^4} = 0.50$$



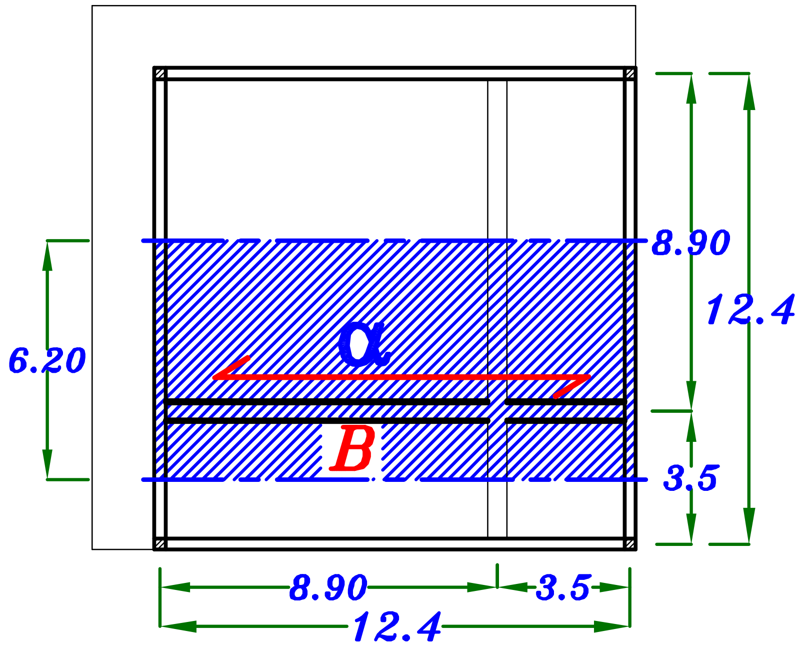
B

$a = 6.20$ m

$$w_1 = w_{av.} * a * \alpha$$

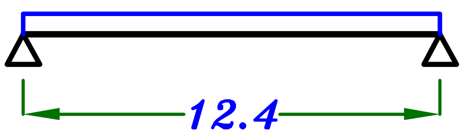
$$= 12.83 * 6.20 * 0.50$$

$$= 39.77 \ kN\backslash m'$$

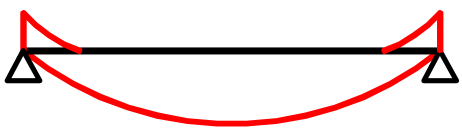


$$M = 39.77 * \frac{12.4^2}{8} = 764.4 \ kN.m$$

$w_1 = 39.77$ kN\m`

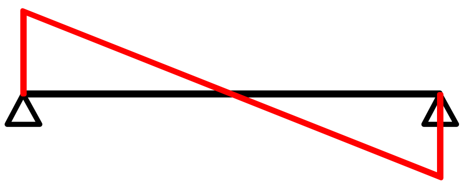


B.M.D.



246.5 kN 764.4 kN.m

S.F.D.



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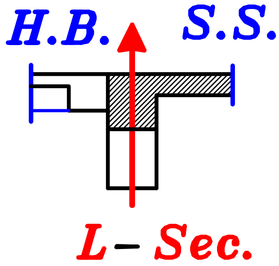
e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 3.5\ m\ ,\quad \frac{L}{2} = 6.2\ m \qquad \theta_{B_1} = \frac{3.5}{6.2} * 90^\circ = 50.80^\circ$$

$$M_1 = 764.4 * \frac{\sin 50.80^\circ}{\sin 90^\circ} = 592.4\ kN.m$$

F – Design the Panelled Beam. B



$\therefore$ Cover $= 70$ mm   Symmetric

$$t = 800\ mm \qquad d = 800 - 70 = 730\ mm$$

$$B = \begin{cases} C.L.-C.L. = 1.75\ m = 1750\ mm \\ 6\,t_s + b = 6 * 150 + 500 = 1400\ mm \\ K\,\frac{L}{10} + b = 1.0 * \frac{12400}{10} + 500 = 1740\ mm \end{cases} \qquad \boxed{B = 1400\ mm}$$

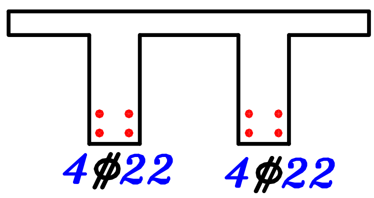
$$730 = C_1 \sqrt{\frac{592.4 * 10^6}{25 * 1400}} \longrightarrow C_1 = 5.61 \longrightarrow J = 0.826$$

$$A_s = \frac{592.4 * 10^6}{0.826 * 360 * 730} = 2729\ mm^2$$

Check $A_{s\,min.}$ $A_{s_{req.}} = 3746.2\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 350 * 1500 = 1640.6\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 3746.2\ mm^2$ $8 \phi 22$



Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (272 \rightarrow 544\ mm^2)$ $6 \phi 10$

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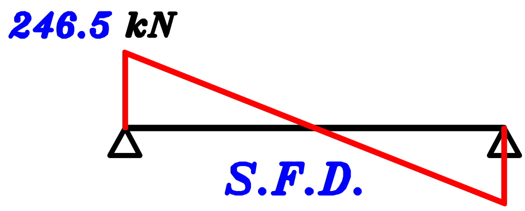
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

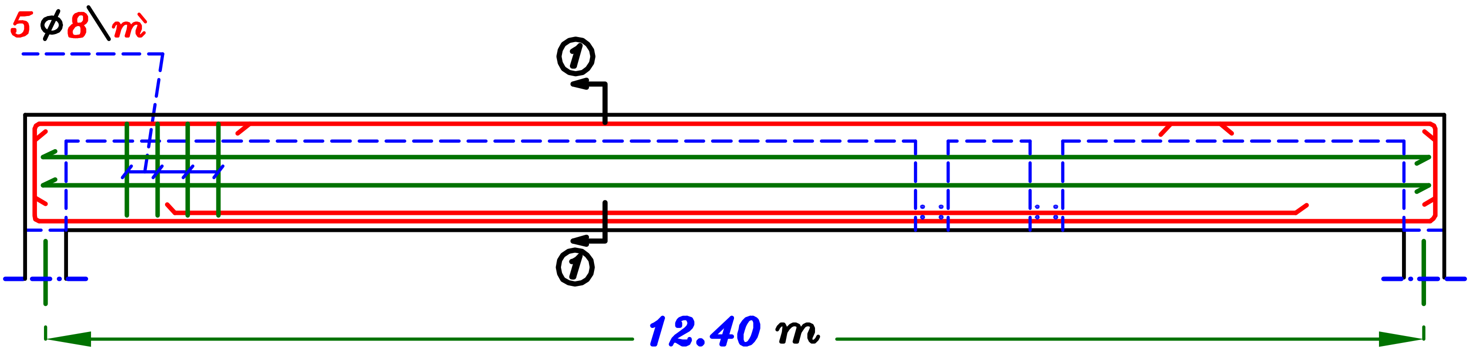
$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$

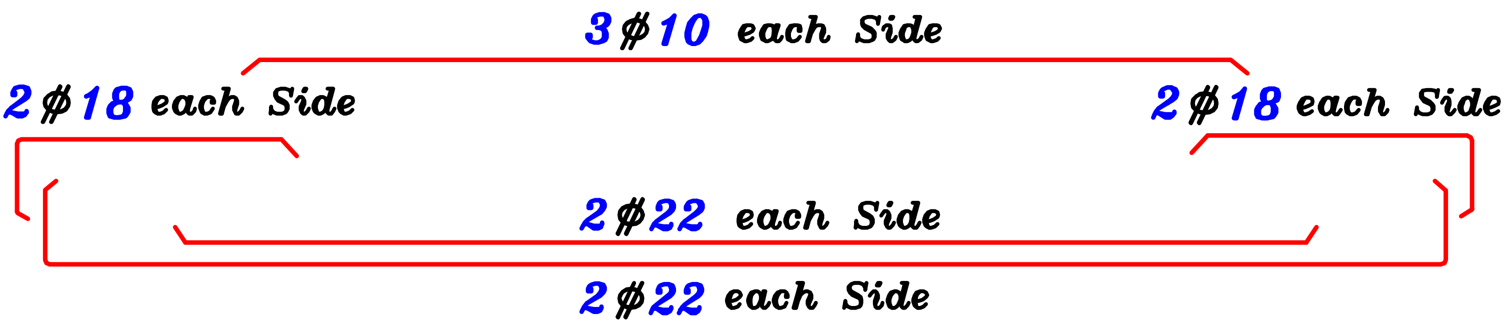


$$q_s = \frac{Q_{max}}{b\,d} = \frac{246.5 * 10^3}{500 * 730} = 0.675 \ N\backslash mm^2 \quad \therefore \ q_s < q_{cu}$$

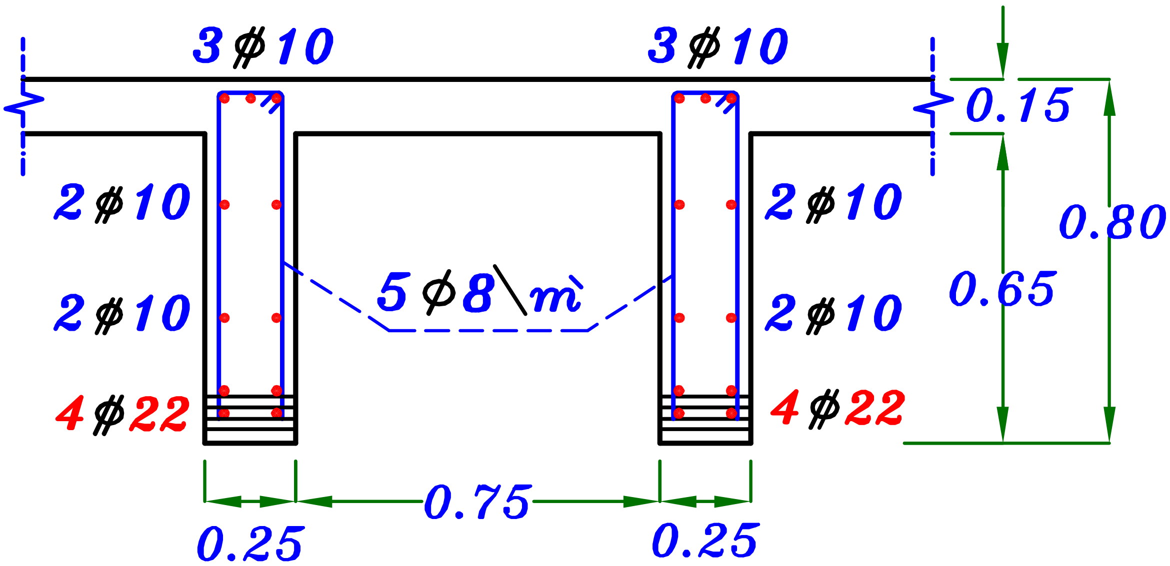
$\therefore$ Use min. Shear RFT. $\quad 5\phi 8\backslash m$

Draw Details of RFT. For the Beam.





Sec. (1–1)



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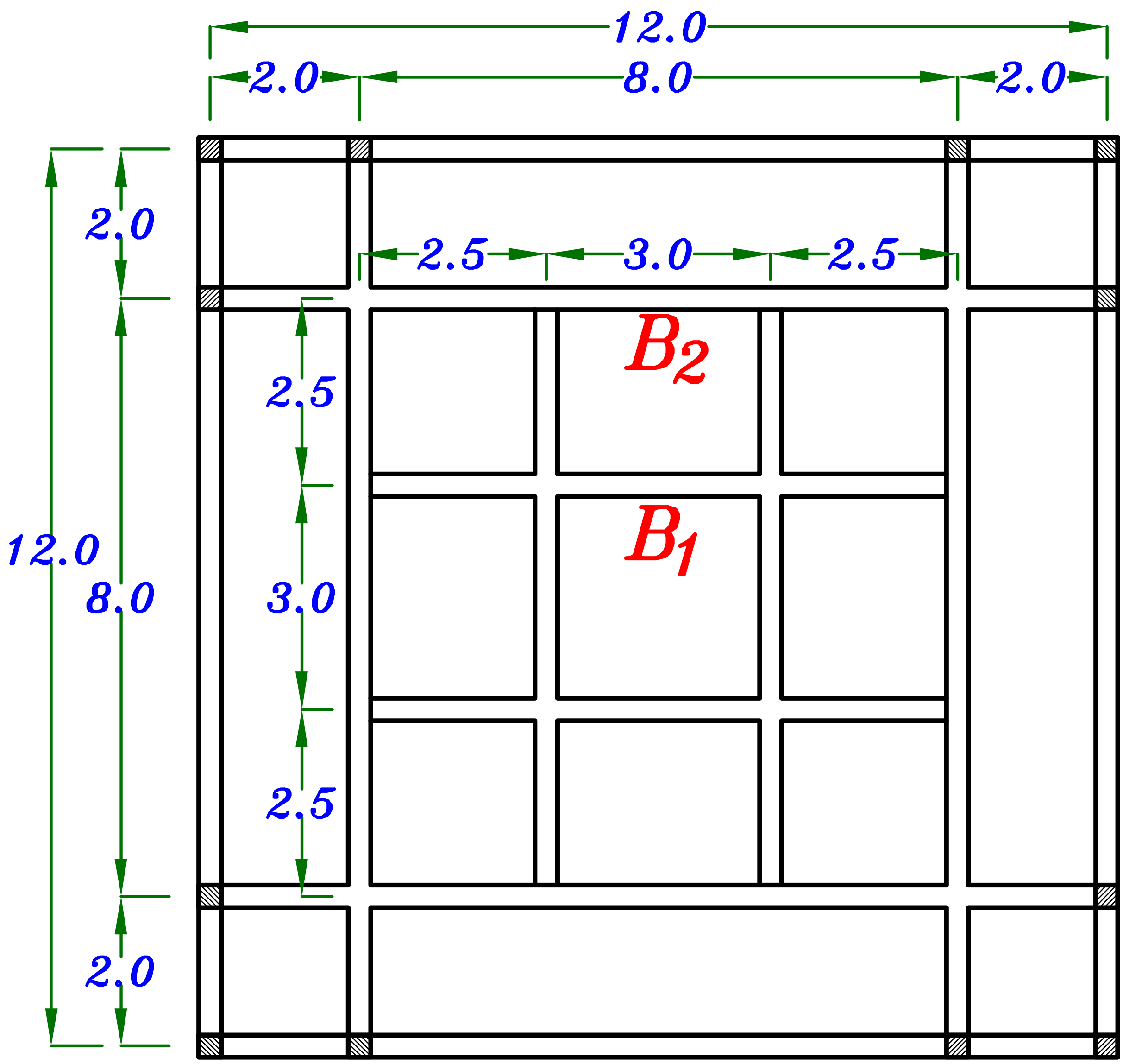
Example.



Data.

$F_{cu} = 25 \ N\backslash mm^2$, $F_y = 360 \ N\backslash mm^2$

$F.C. = 1.5 \ kN\backslash m^2$, $L.L. = 2.0 \ kN\backslash m^2$

Req.

① Draw a sketch For the reinforcement of slabs in plan.

② Design the panelled Beams B_1 , B_2
& Draw Details of RFT. in elevation.

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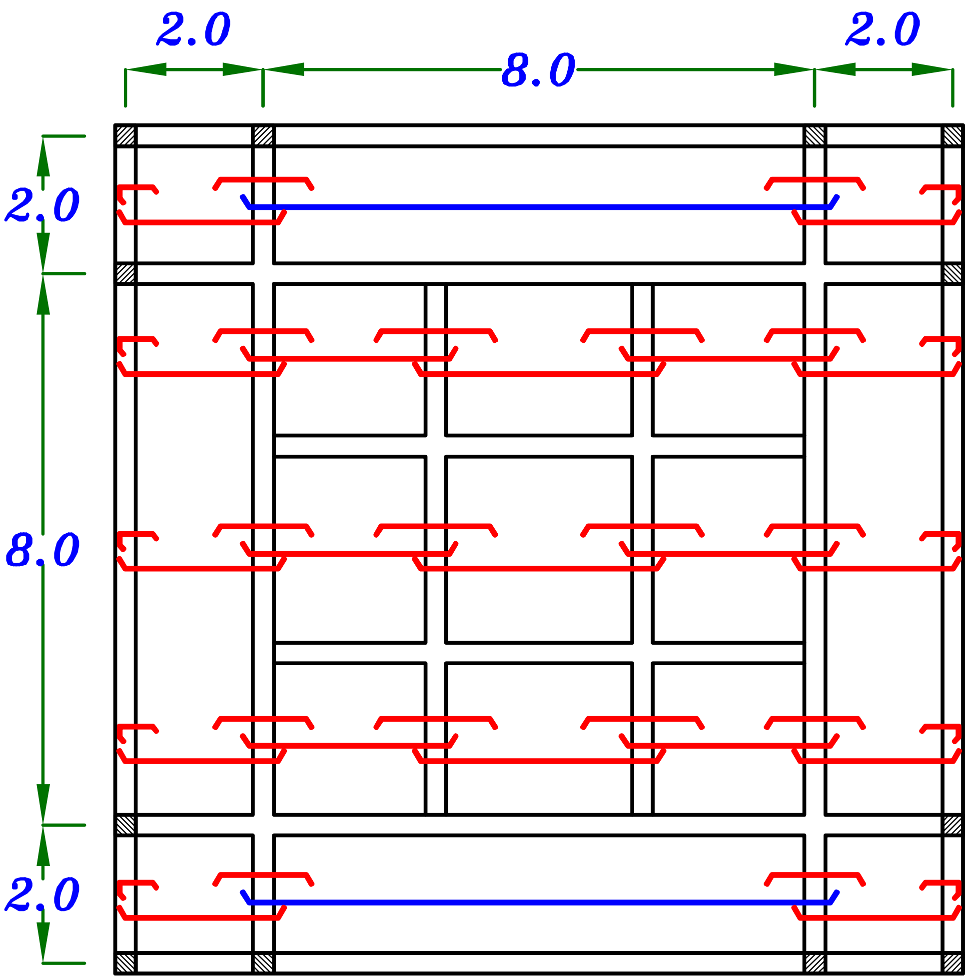
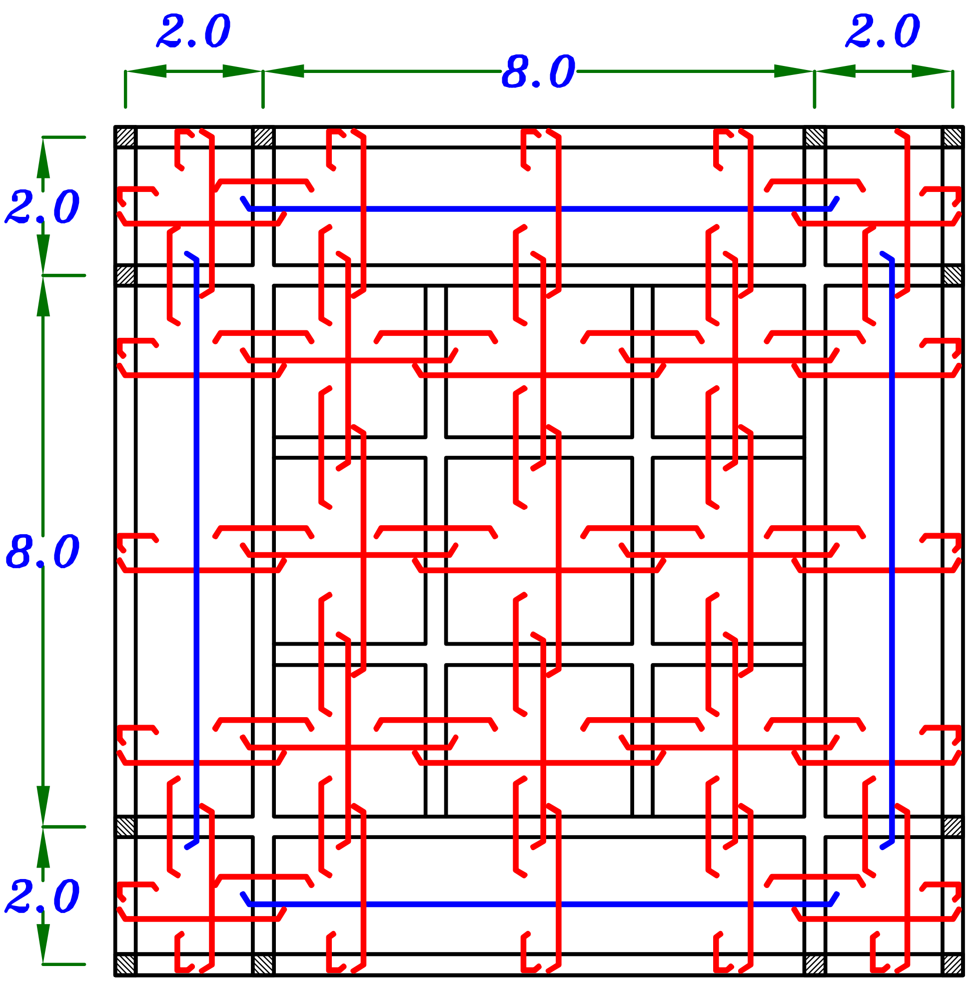
Sketch For the reinforcement of slabs in plan.



تسليح البلاطات
فى اتجاه واحد فقط



تسليح البلاطات
فى الاتجاهين

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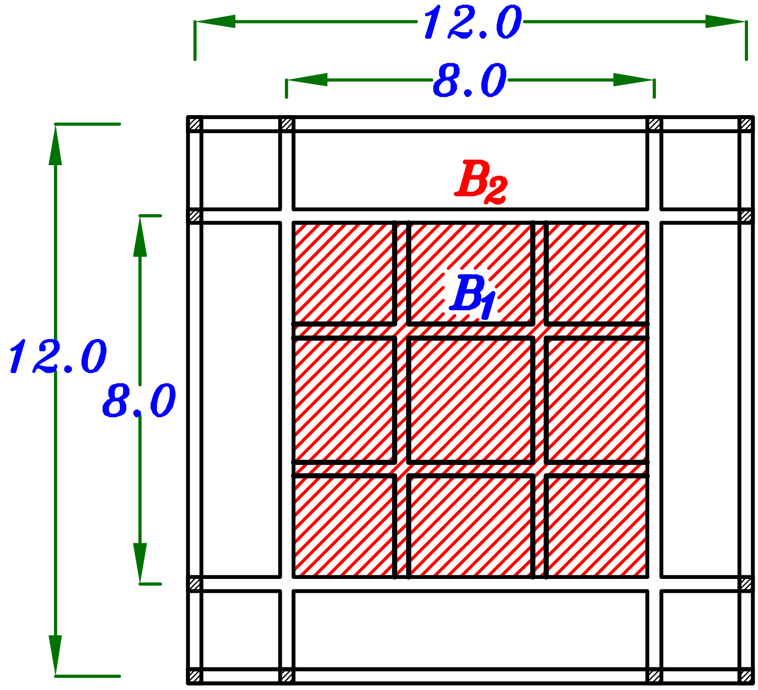
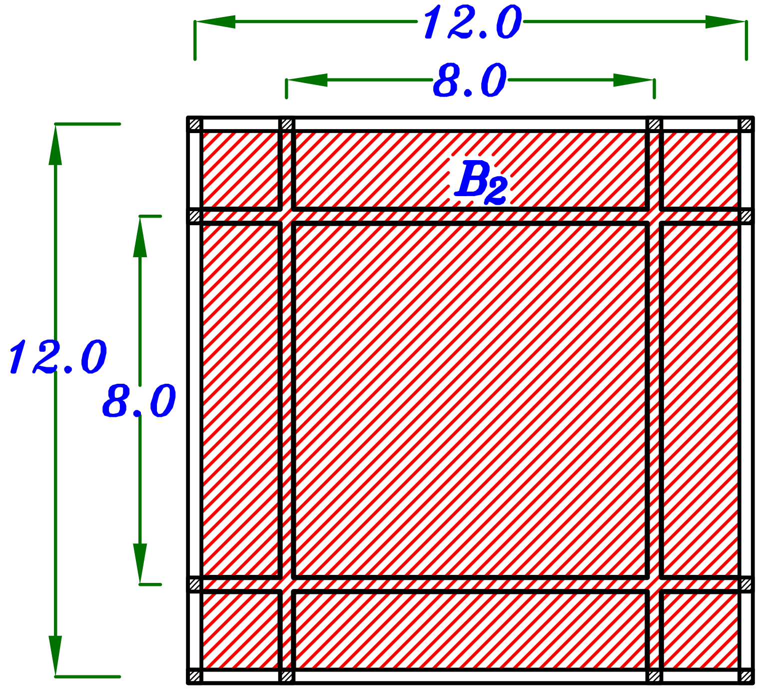
فكره المسأله .

نصمم اولا شبكه الكمرات B_1 حيث تكون مساحه الشبكه $(8.0*8.0)$



ثم نصمم شبكه الكمرات B_2 حيث تكون مساحه الشبكه $(12.0*12.0)$



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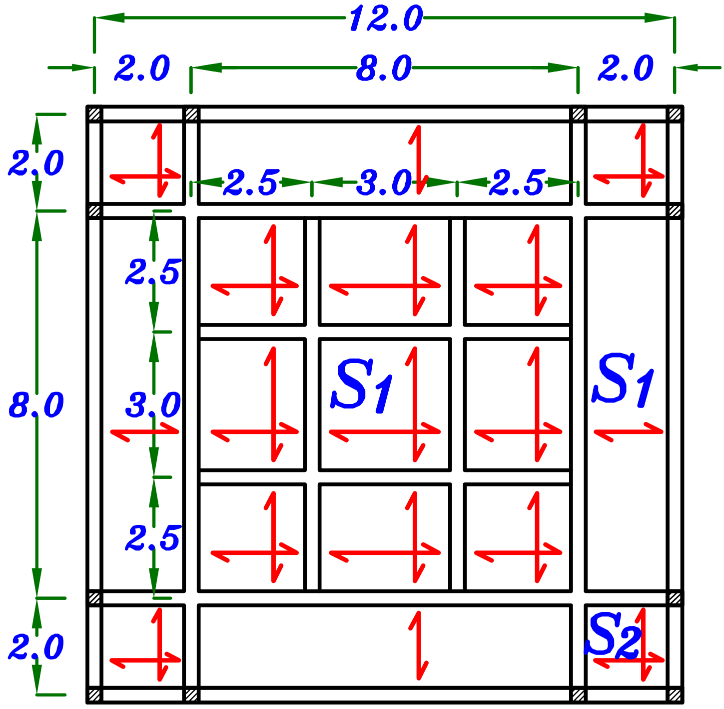
a – Choose the Thickness of the Slab. (t_s)



S_1 two way $L_s = 3.0\ m$

$$t_s = \frac{3000}{45} = 66.6 \text{ mm}$$

S_2 two way $L_s = 2.0\ m$

$$t_s = \frac{2000}{40} = 50.0 \text{ mm}$$

S_3 One way $L_s = 2.0\ m$

$$t_s = \frac{2000}{30} = 66.6 \text{ mm}$$

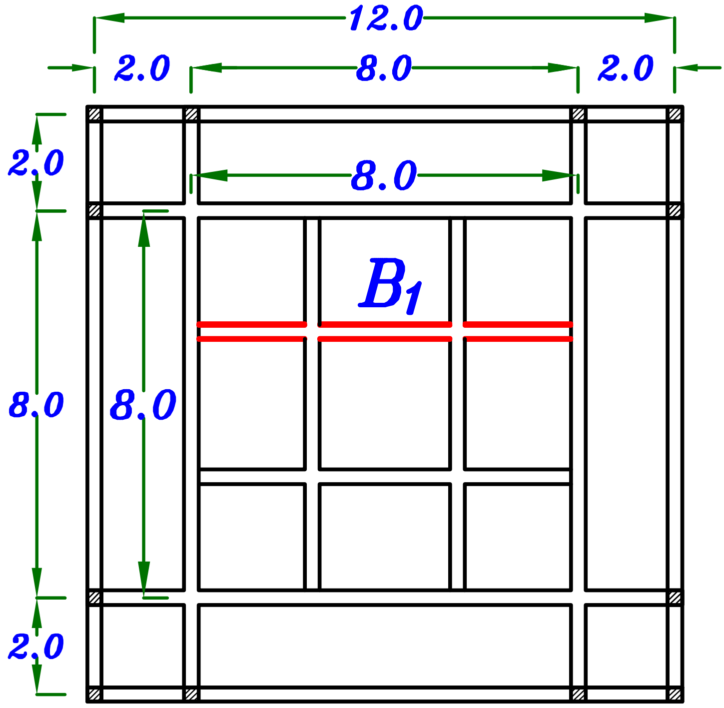
Take (t_s) the bigger value $\boxed{t_s = 80 \text{ mm}}$

b – Get the Loads on the Slab (w_s).

$$w_s = 1.4\left(t_s\,\gamma_c + F.C.\right) + 1.6\ L.L.$$

$$w_s = 1.4\left(0.08 * 25 + 1.5\right) + 1.6\,(2.0) = 8.10 \text{ kN}\backslash\text{m}^2$$

Design the Panelled Beam. B_1



a – Get the Dimensions of the beam. (b, t)

Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{8.0}{16} = 0.50 \text{ m}$$

$\boxed{b = 0.25\ m}$ $\boxed{t = 0.50\ m}$

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b - Get the Loads on the Slab. (w_{av})

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L_s}$$

$$w_{av.} = w_s + \frac{1.4 * b\,(t - t_s)\left[\text{مجموع أطوال الكمرات الداخلية}\right] * \gamma_c}{L * L_s}$$

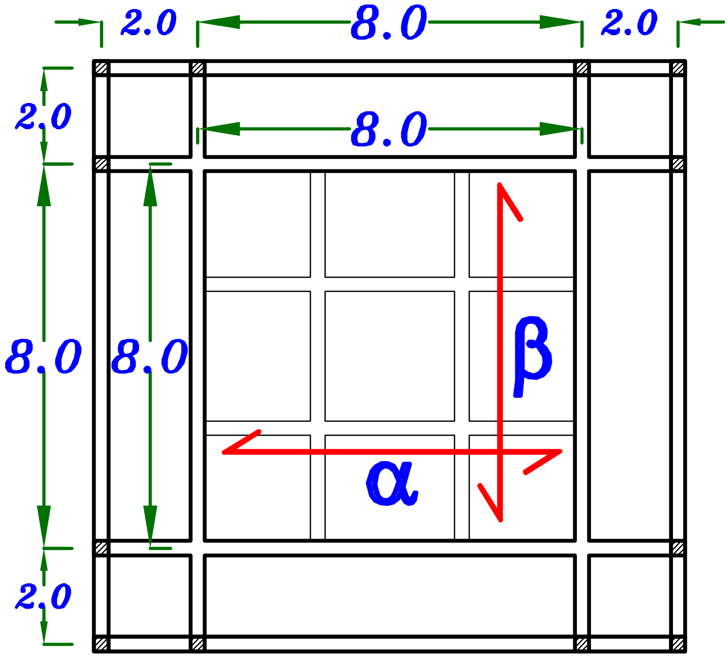
$$w_{av.} = 8.10 + \frac{1.4 * 0.25\,(0.5 - 0.08)\,[2*8 + 2*8] * 25}{8.0 * 8.0} = 9.93 \quad kN\backslash m^2$$

c - Calculate α , β By using Grashoff.

$$r = \frac{m\,L}{\grave{m}\,L_s} = \frac{(1.0)\,8.0}{(1.0)\,8.0} = 1.0$$

$$\alpha = \frac{r^4}{1 + r^4} = \frac{(1.0)^4}{1 + (1.0)^4} = 0.5$$

$$\beta = \frac{1}{1 + r^4} = \frac{1}{1 + (1.0)^4} = 0.5$$



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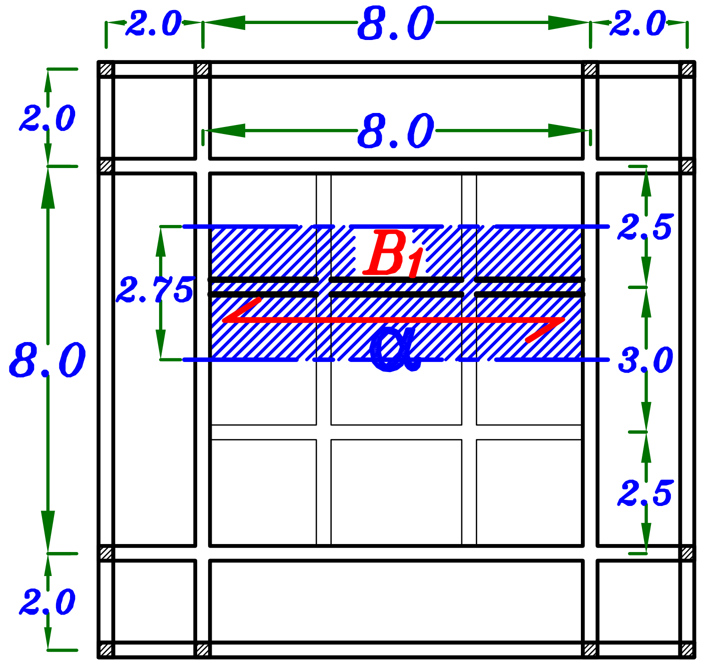
d- Get the Loads on the Panelled Beam & Calculate the B.M.



$$a = 2.75\ m$$

$$w_1 = w_{av.} * a * \alpha$$

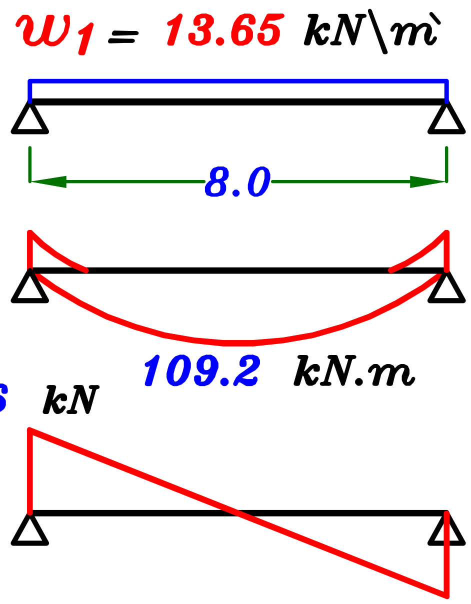
$$= 9.93 * 2.75 * 0.50$$

$$= 13.65\ kN\backslash m`$$

$$M = 13.65 * \frac{8.0^2}{8} = 109.2\ kN.m$$

B.M.D.

S.F.D.



e - Calculate the reduction Factor of the B.M. $\left(\frac{Sin\ \theta}{Sin\ 90}\right)$

$$X = 2.5\ m\ ,\ \frac{L}{2} = 4.0\ m$$

$$\theta_{B_1} = \frac{2.5}{4.0} * 90^\circ = 56.25^\circ$$

$$M_1 = 109.2 * \frac{\sin 56.25^\circ}{\sin 90^\circ} = 90.79\ kN.m$$

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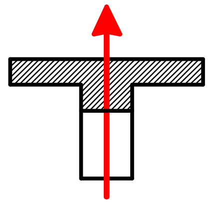
F- Design the Panelled Beam. B_1

$\therefore$ Cover = 70 mm Symmetric

$t = 500$ mm $d = 500 - 70 = 430$ mm



T- Sec.

$$B = \begin{cases} C.L.-C.L. = 2.75 \text{ m} = 2750 \text{ mm} \\ 16\, t_s + b = 16 * 80 + 250 = 1530 \text{ mm} \\ K \frac{L}{5} + b = 1.0 * \frac{8000}{5} + 250 = 1850 \text{ mm} \end{cases}$$

$$\boxed{B = 1530 \text{ mm}}$$

$$430 = C_1 \sqrt{\frac{90.79 * 10^6}{25 * 1530}} \longrightarrow C_1 = 8.82 \longrightarrow J = 0.826$$

$$A_s = \frac{90.79 * 10^6}{0.826 * 360 * 430} = 710.0 \text{ mm}^2$$

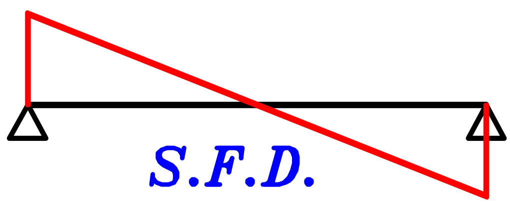
Check $A_{s\,min.}$ $A_{s_{req.}} = 710.0$ mm^2

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 430 = 335.9 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 710.0$ mm^2 **$4 \phi 16$**

Stirrup Hangers = $(0.1 \rightarrow 0.2)\, A_s = (71 \rightarrow 142 \text{ mm}^2)$ **$2 \phi 10$**

Check Shear.



$$q_{cu} = (0.24) \sqrt{\frac{25}{1.50}} = 0.98 \text{ N\textbackslash mm}^2$$

$$q_{u_{max}} = (0.70) \sqrt{\frac{25}{1.50}} = 2.85 \text{ N\textbackslash mm}^2$$

$$q_s = \frac{Q_{max}}{b\, d} = \frac{54.6 * 10^3}{250 * 430} = 0.507 \text{ N\textbackslash mm}^2 \quad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. **$5 \phi 8 \backslash m'$**

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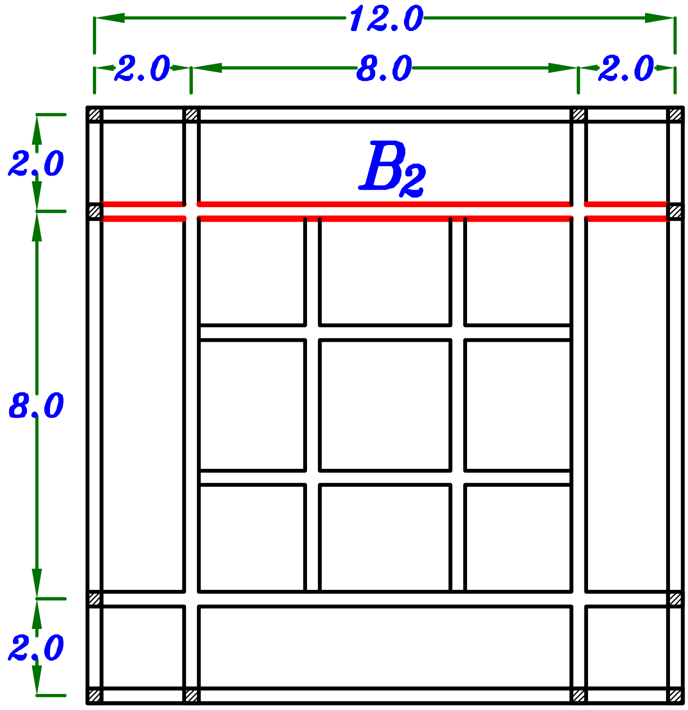
Design the Panelled Beam. B_2

a – Get the Dimensions of the beam. (b, t)

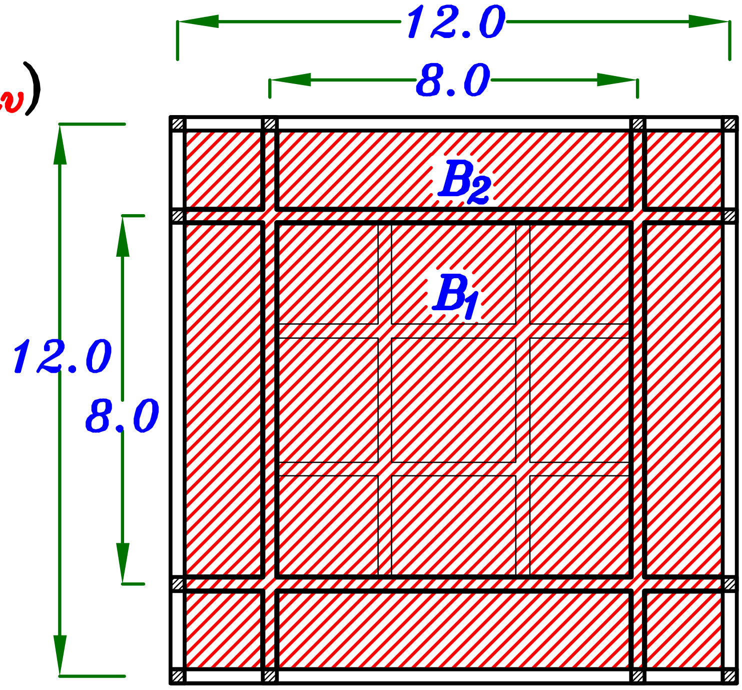


Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{12.0}{16} = 0.75\ m$$

$$\boxed{b = 0.25\ m} \qquad \boxed{t = 0.75\ m}$$

– Get the Loads on the Slab. (w_{av})



$$w_{av.} = \frac{\sum Weight}{Area}$$

$$w_{av.} = \frac{\text{Weight of slabs} + \text{Total Weight of Panelled Beams } B_1, B_2}{L * L_s}$$

$$w_{av.} = \frac{w_s * (L * L_s) + 1.4 * b(t - t_s)\,[B_1 \text{ مجموع أطوال الكمرات}] * \gamma_c + 1.4 * b(t - t_s)\,[B_2 \text{ مجموع أطوال الكمرات}] * \gamma_c}{L * L_s}$$

$$w_{av.} = \frac{8.10\,(12 * 12) + 1.4 * 0.25(0.5 - 0.08)\,[2*8 + 2*8] * 25 + 1.4 * 0.25(0.75 - 0.08)\,[2*12 + 2*12] * 25}{12.0 * 12.0}$$

$$= 10.87\ kN \backslash m^2$$

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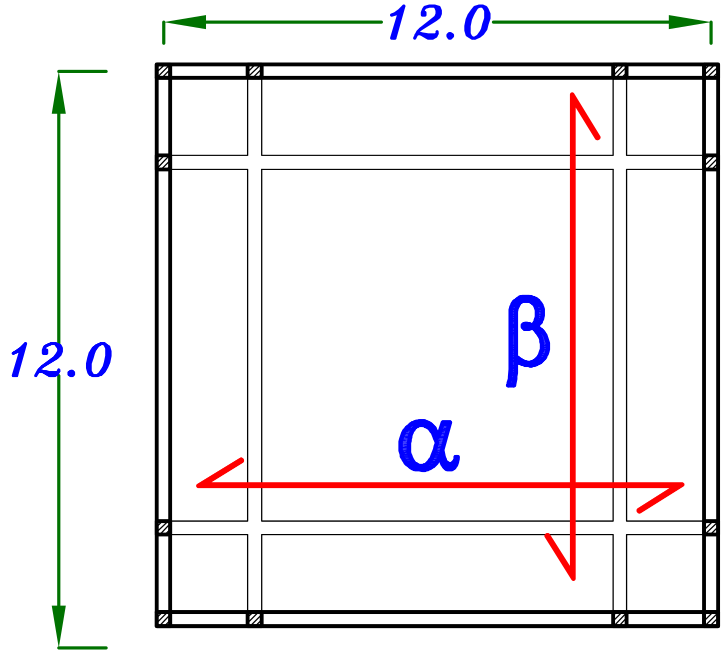
c – Calculate α , β By using Grashoff.

$$r = \frac{m\,L}{\grave{m}\,L_s} = \frac{(1.0)\,12.0}{(1.0)\,12.0} = 1.0$$

$$\alpha = \frac{r^4}{1+r^4} = \frac{(1.0)^4}{1+(1.0)^4} = 0.5$$

$$\beta = \frac{1}{1+r^4} = \frac{1}{1+(1.0)^4} = 0.5$$



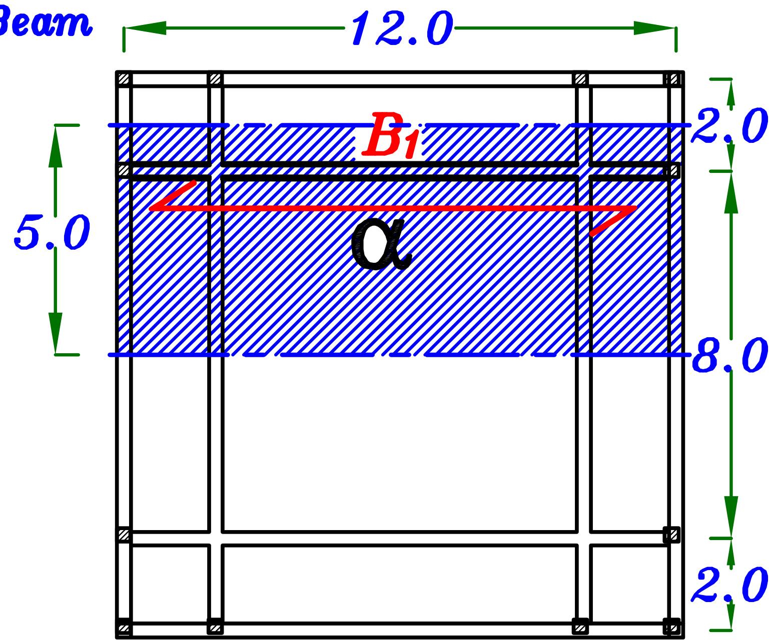
d– Get the Loads on the Panelled Beam & Calculate the B.M.

$$a = 5.0 \quad m$$

$$w_1 = w_{av.} * a * \alpha$$

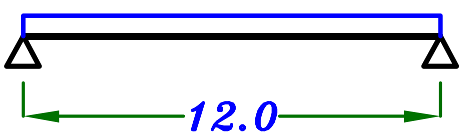
$$= 10.87 * 5.0 * 0.50$$

$$= 27.175 \ kN\backslash \grave{m}$$

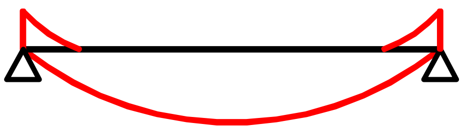


$$M = 27.175 * \frac{12.0^2}{8} = 489.15 \ kN.m$$

$w_1 = 27.175 \ kN\backslash \grave{m}$

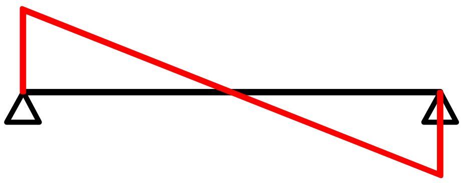


B.M.D.



163.05 kN   489.15 kN.m

S.F.D.



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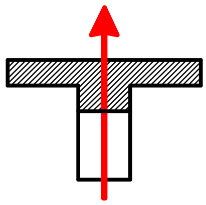
e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 2.0\ m\ ,\quad \frac{L}{2} = 6.0\ m \qquad \theta_{B_2} = \frac{2.0}{6.0} * 90^\circ = 30^\circ$$

$$M_2 = 489.15 * \frac{\sin 30^\circ}{\sin 90^\circ} = 244.57\ kN.m$$

F– Design the Panelled Beam. B_2

$\therefore$ Cover = 70 mm   Symmetric



T – Sec.

$t = 750\ mm \qquad d = 750 - 70 = 680\ mm$

$$B = \begin{cases} C.L.-C.L. = 5.0\ m = 5000\ mm \\ 16\,t_s + b = 16 * 80 + 250 = 1530\ mm \\ K\frac{L}{5} + b = 1.0 * \frac{12000}{5} + 250 = 2650\ mm \end{cases} \qquad \boxed{B = 1530\ mm}$$

$$680 = C_1 \sqrt{\frac{244.57 * 10^6}{25 * 1530}} \longrightarrow C_1 = 8.50 \longrightarrow J = 0.826$$

$$A_s = \frac{244.57 * 10^6}{0.826 * 360 * 680} = 1209.5\ mm^2$$

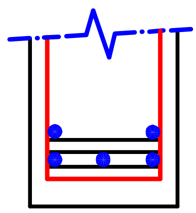
Check $A_{s_{min.}}$ $A_{s_{req.}} = 1209.5\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 680 = 531.25\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1209.5\ mm^2$ 5 ϕ 18

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (120 \rightarrow 240\ mm^2)$

3 ϕ 10



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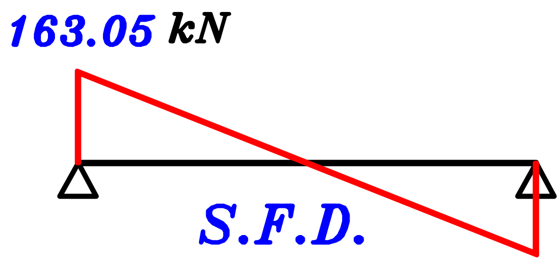
Check Shear.

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.50}} = 0.98 \ N\backslash mm^2$$

$$q_{u_{max}} = (0.70)\sqrt{\frac{25}{1.50}} = 2.85 \ N\backslash mm^2$$



$$q_s = \frac{Q_{max}}{b\ d} = \frac{163.05 * 10^3}{250 * 680} = 0.96 \ N\backslash mm^2 \quad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $5\phi 8\backslash m'$

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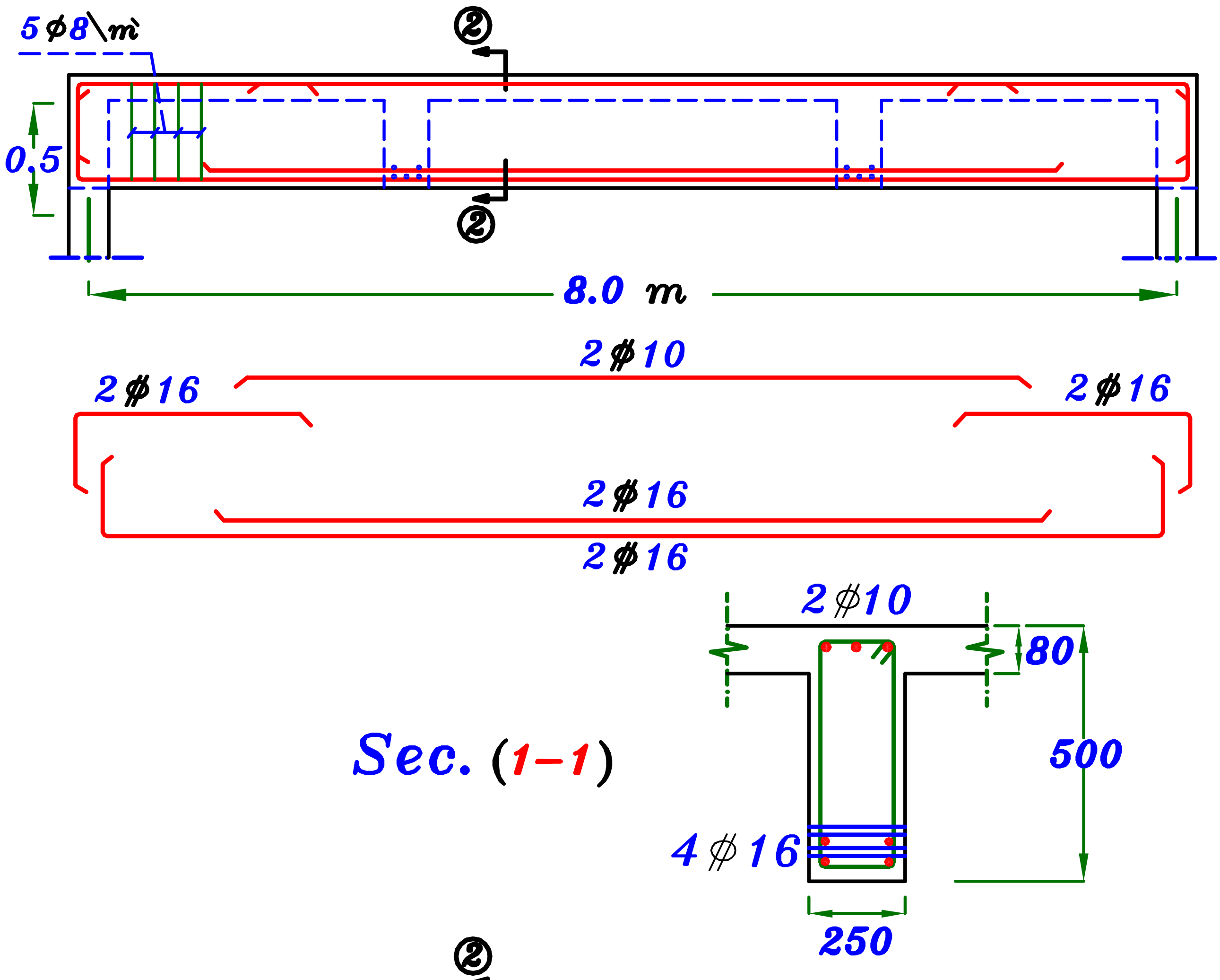
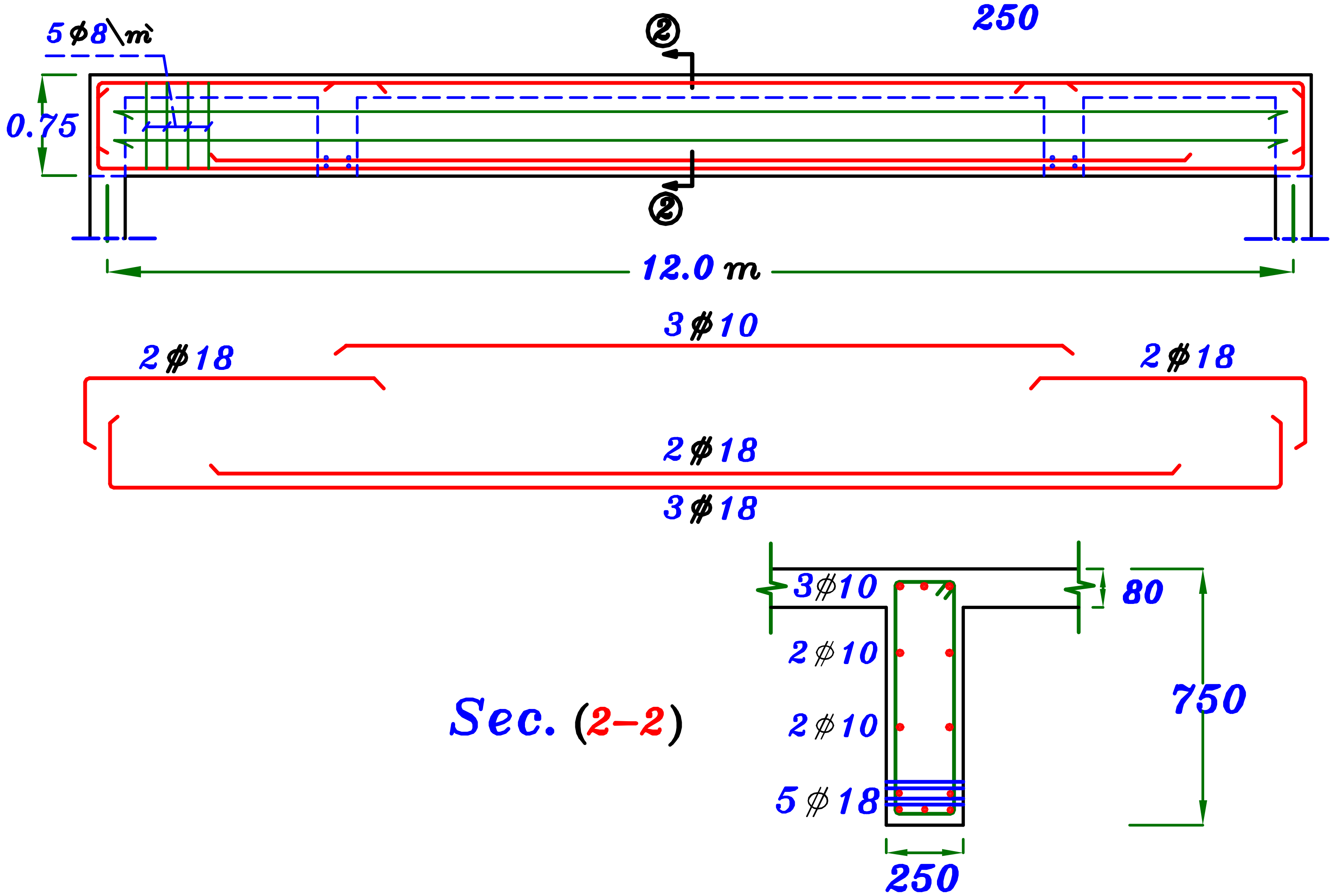
g – Draw Details of RFT. For the Beams.

B1



Sec. (1–1)

B2



Sec. (2–2)

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Example.

Data to be used in all the exam.

$F_{cu} = 25$ MPa , $F_y = 420$ MPa , F.C. = 3.0 KPa , L.L. = 1.0 KPa

Block (200 * 400 * 200)   Weight of Block. = 170 N

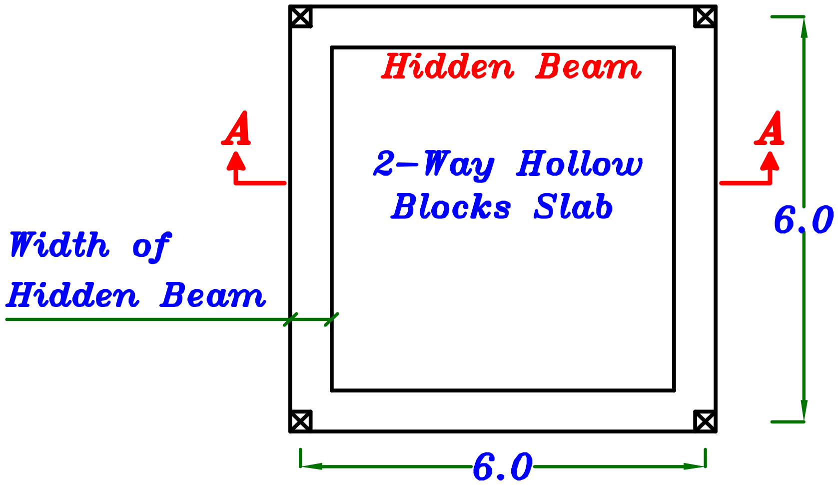
For the given reinforced concrete hall shown in Fig.(1) It is required to :

1 – Design all slabs at level (+3.00) and (+6.00).

2 – Draw Details of reinforcement in quarter plan (scale 1:50) and cross section (scale 1:25)

3 – Design the panelled beams at level(+3.00).

4 – Draw Details of reinforcement of one of the panelled beams in elevation(scale 1:50) and cross section (scale 1:25).



Plan at level (+6.00)

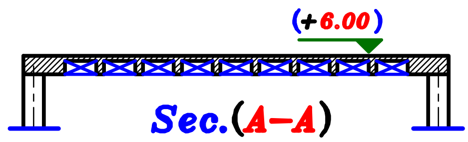
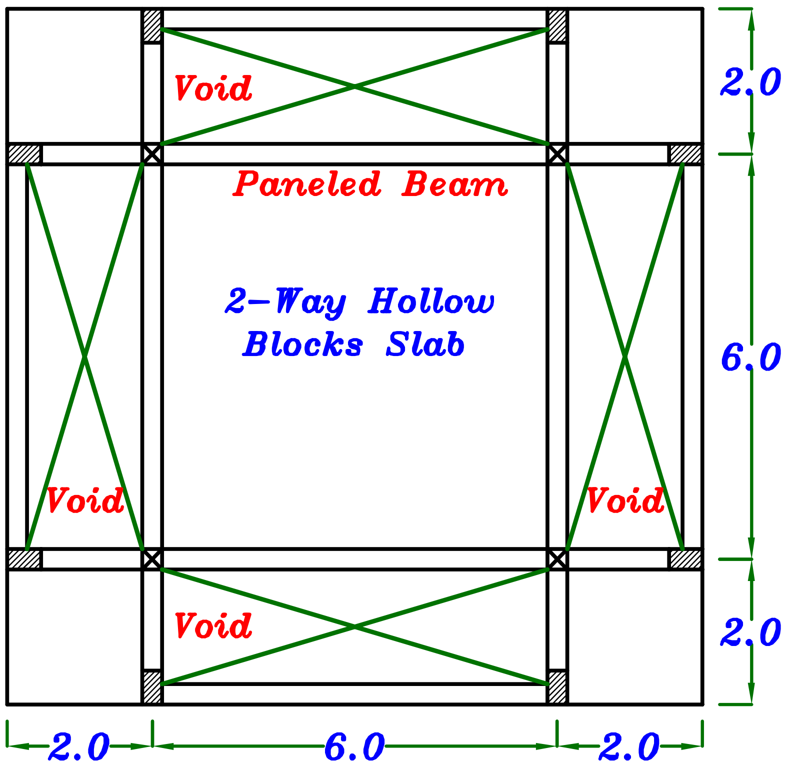


Fig. (1)



Plan at level (+3.00)

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1 – Design all slabs at level (+3.00) and (+6.00).

1 – Solid Slabs.

$$t_s = \frac{2000}{10} = 200 \text{ mm} \qquad \boxed{t_s = 200 \text{ mm}}$$

$$w_s = 1.4\,(0.20*25 + 3.0) + 1.6\,(1.0) = 12.80 \text{ kN}\backslash m^2$$

2 – Two way Hollow Blocks.

$$b = 0.1 \text{ m} \qquad e = 0.4 \text{ m}$$

$$S = e + b = 0.4 + 0.1 = 0.5 \text{ m}$$

$$\boxed{\begin{aligned} W_{ribT} &= \left[1.4\,(t_s \gamma_c + F.C.) + 1.6\,(L.L.)\right](S*S) \\ &+ 1.4*b\,h*(2S - b)*\gamma_c + 1.4*(\text{وزن ال Block})\left(\frac{e}{a}\right) \end{aligned}}$$

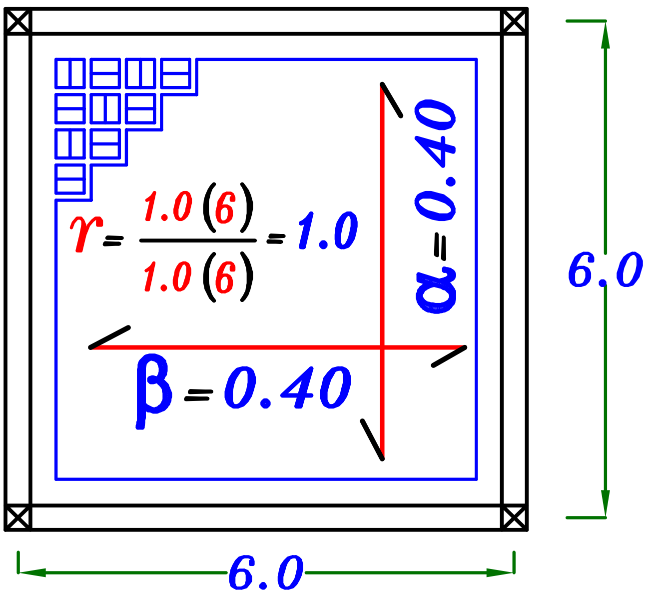
$$\therefore W_{ribT} = \left[1.4\,(0.05*25 + 3.0) + 1.6\,(1.0)\right](0.5*0.5)$$

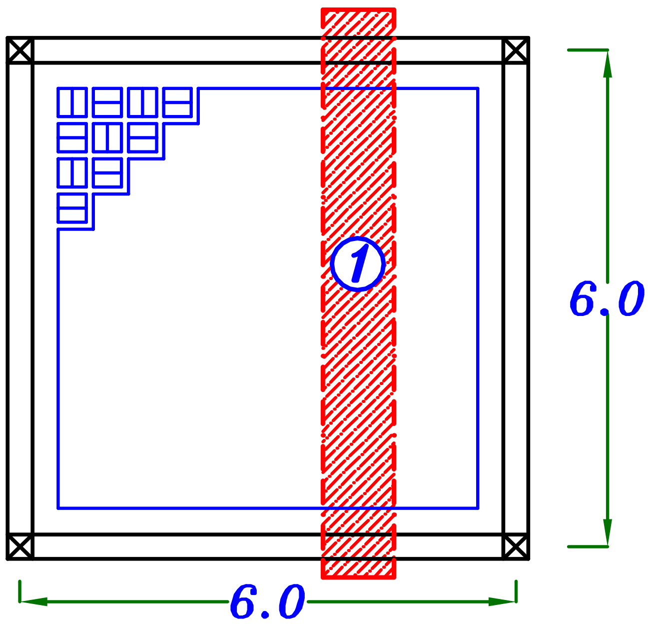
$$+ 1.4\,(0.1*0.20*(2*0.5 - 0.1)*25) + 1.4\left(\frac{170}{1000}\right)\left(\frac{0.4}{0.2}\right) = 2.99$$

$$(kN \backslash (S*2S))$$

$$W_{rib} = \frac{W_{ribT}}{S} = \frac{2.99}{0.5} = 5.98 \text{ kN} \backslash (S*m)$$

Two way Hollow Blocks slabs at levels (+3.00) and (+6.00). are the same.





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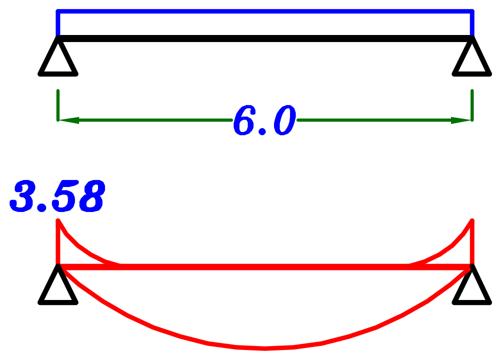
Strip ①

$\alpha w_{rib} = 0.40\,(5.98) = 2.39\ kN\backslash m$



$M = 10.75\ kN.m\backslash rib$

$d = t - 40\ mm = 250 - 40 = 210\ mm$

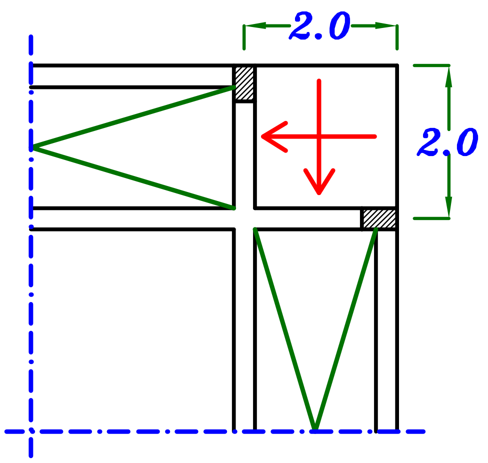
$\therefore\ 210 = C_1 \sqrt{\dfrac{10.75 * 10^6}{25 * 500}}$

$\rightarrow C_1 = 7.16 \qquad \rightarrow J = 0.826$

$$A_s = \frac{M}{J\,F_y\,d} = \frac{10.75 * 10^6}{0.826 * 420 * 210} = 147.5\ mm^2\backslash rib$$

$\boxed{2\phi 10\backslash rib}$

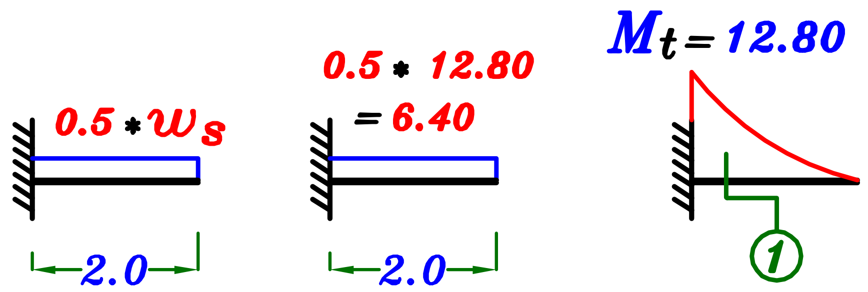
Cantilever Solid Slab.

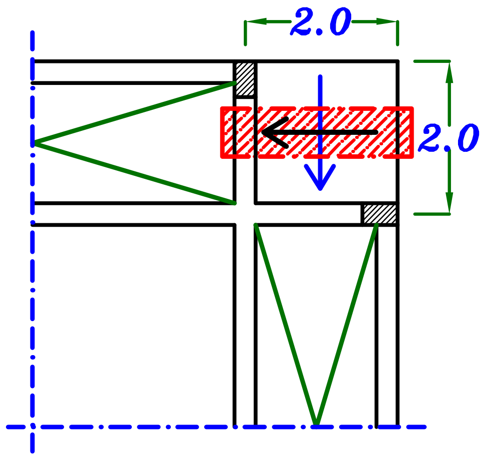


يتوزع حمل البلاطه على ال two cantilevers

أى يتوزع الحمل فى الاتجاهين كل اتجاه $0.5\ w_s$

تعمل Torsion على الكمره





Sec. ① $\quad M_{U.L.} = 12.80\ kN.m\backslash m$

$t_s = 200\ mm\ ,\ d = 200 - 20 = 180\ mm$

$180 = C_1 \sqrt{\dfrac{12.80 * 10^6}{25 * 1000}} \rightarrow C_1 = 7.95 \rightarrow J = 0.826$

$A_s = \dfrac{12.80 * 10^6}{0.826 * 420 * 180} = 204.9\ mm^2/m$ $\quad$ $5\phi 10\backslash m$

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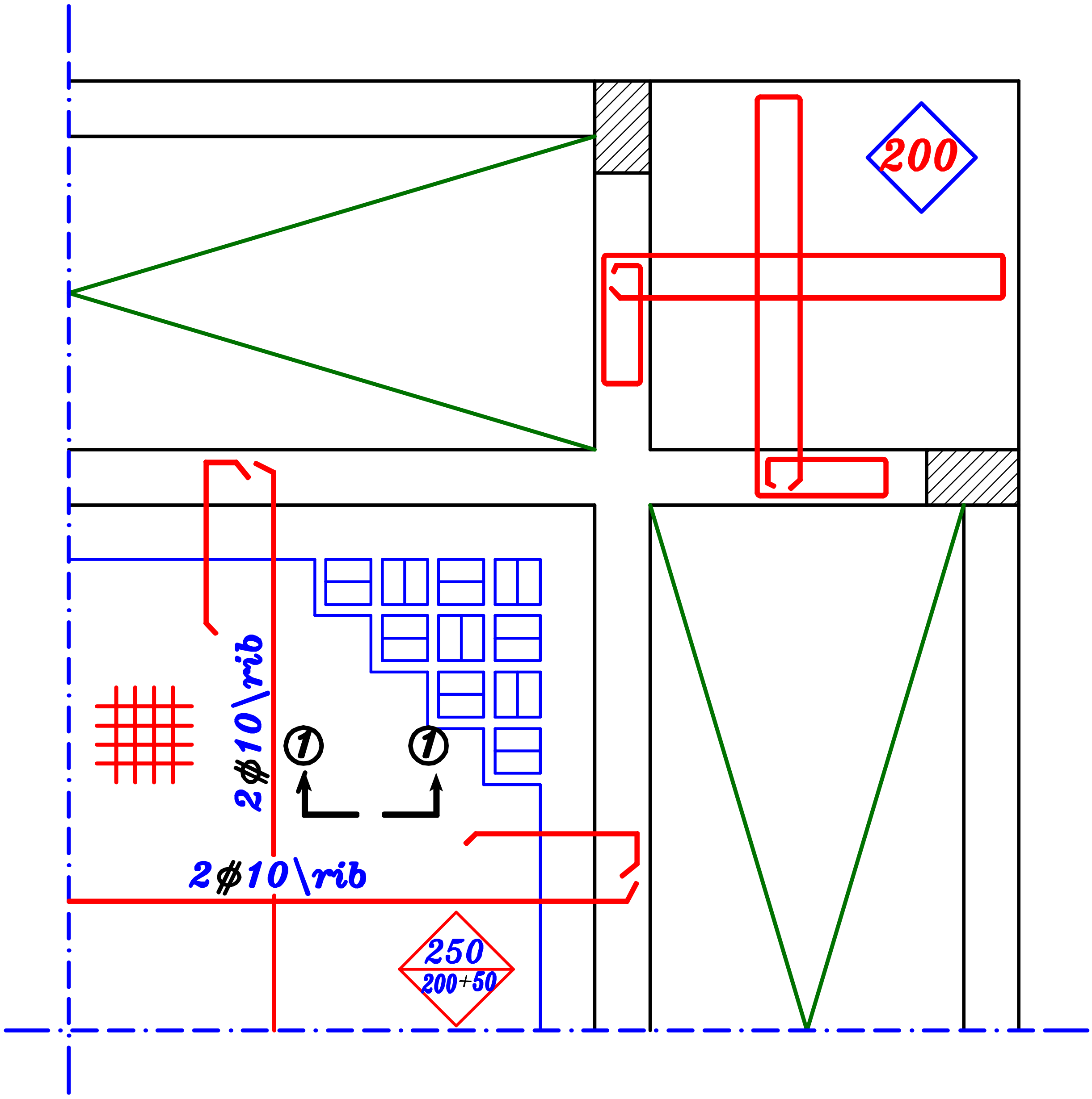
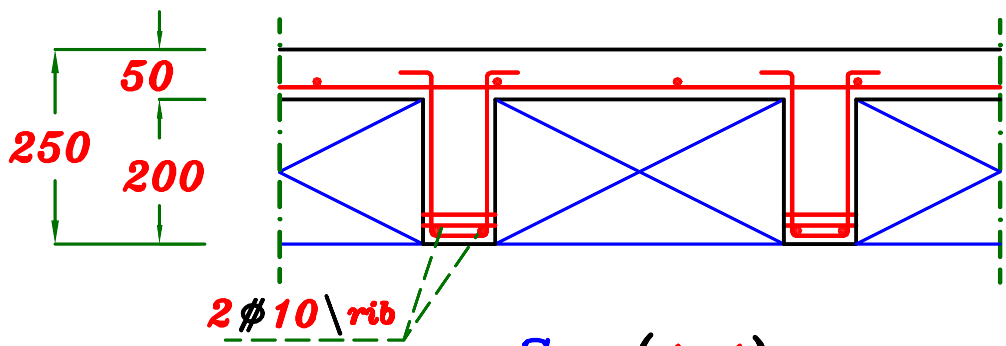
2 – *Draw Details of reinforcement in quarter plan (scale 1:50) and cross section (scale 1:25)*





Sec. (1–1)

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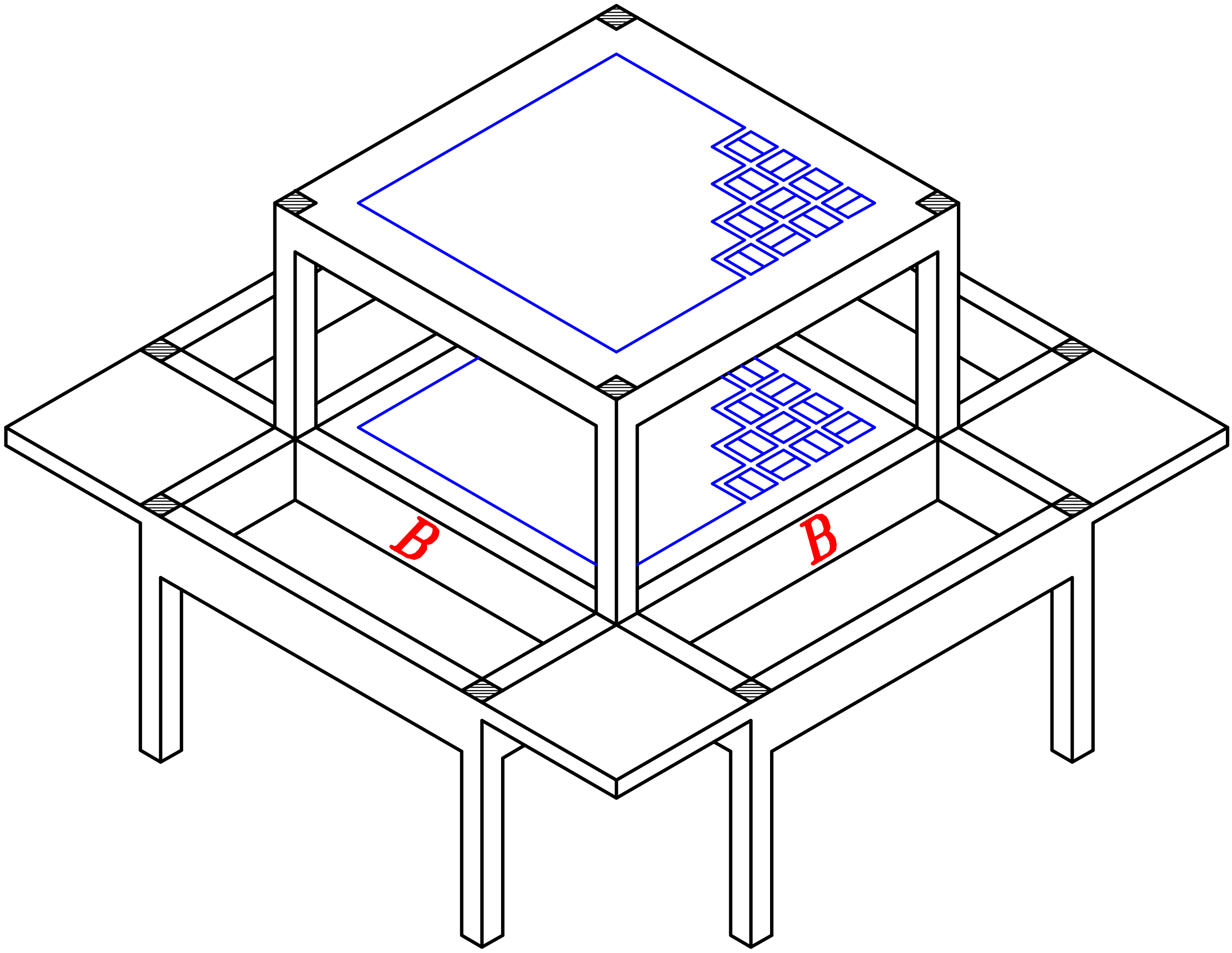
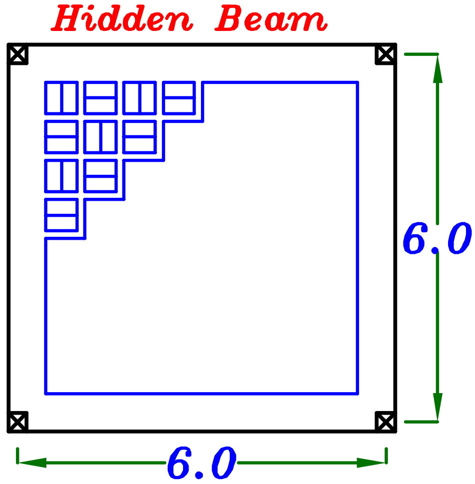
3- Design the panelled beams at level (+3.00).



Get the total weight For the upper plan.

Take O.W.(Hidden Beam) = 6.0 kN\m` (U.L.)

$$Post = 0.25 * 0.25 * 3.0 * 25 * 1.4 = 6.5 \ kN$$



$$Total\ Weight = \left(\frac{w_{rib}}{S}\right) * H.B.\ area + o.w._{(Beam)} * Length_{(Beams)} + 4\ Posts$$

$$Total\ Weight = \left(\frac{5.98}{0.5}\right)(6*6) + 6.0\,(6*4) + 4*6.5 = 600.56 \ kN$$

$$Load\ From\ one\ Post\ (P) = \frac{600.56}{4} = 150.14 \ kN \quad (U.L.)$$

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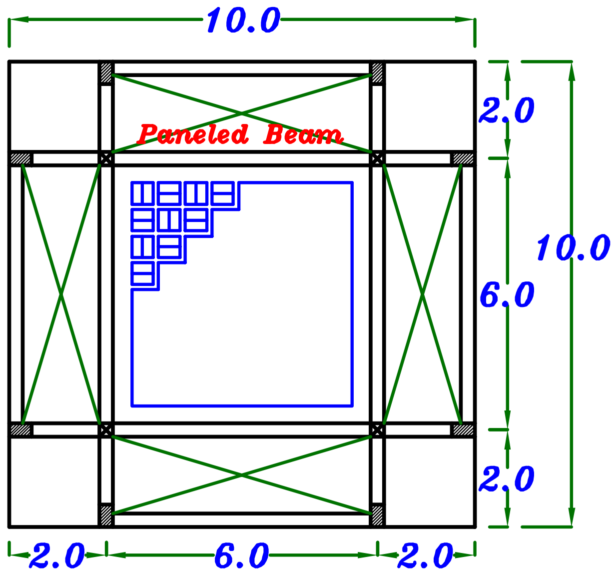
a- Get the Dimensions of the beam. (b, t)

Take $b = 0.25\ m.$

$$t = \frac{10}{16} = 0.65\ m.$$



b - Get the Loads on the Slab. (w_{av})

$$(w_{av.})_{U.L.} = \frac{w_s * area(s.s.) + \left(\frac{w_{rib}}{S}\right) * area(H.B.) + Panelled\ Beams}{Total\ area}$$

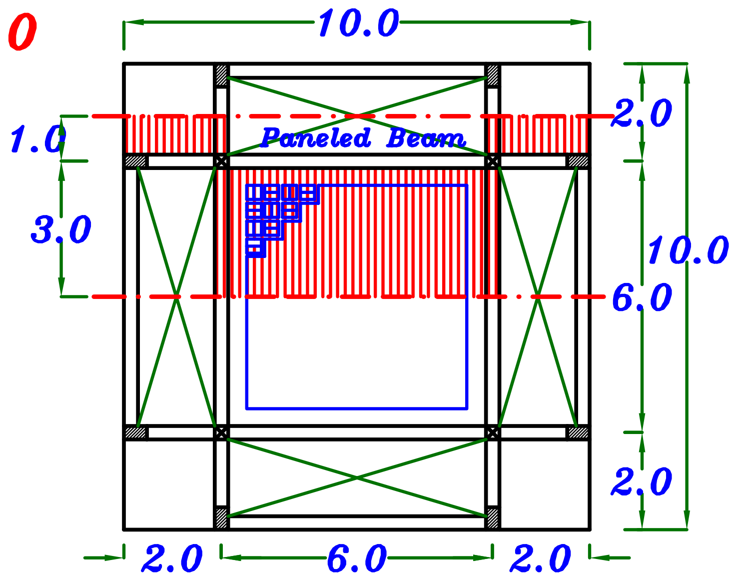
$$= \frac{12.80 * 4(2*2) + \left(\frac{5.98}{0.5}\right)(6*6) + 1.4\left[0.25(0.65 - 0.20)(4*10)*25\right]}{10*10 - 4(2*6)}$$

$$= 15.25\ kN\backslash m^2$$

C - Calculate α , β By using Grashoff.

$$r = \frac{m\ L}{\grave{m}\ L_s} = \frac{(1.0)\ 10}{(1.0)\ 10} = 1.0$$

$$\alpha = \beta = 0.50$$



$$a_1 = 1.0\ m$$

$$a_2 = 3.0\ m$$

$$\alpha P = \beta P = 0.50 * 150.14 = 75.07\ kN$$

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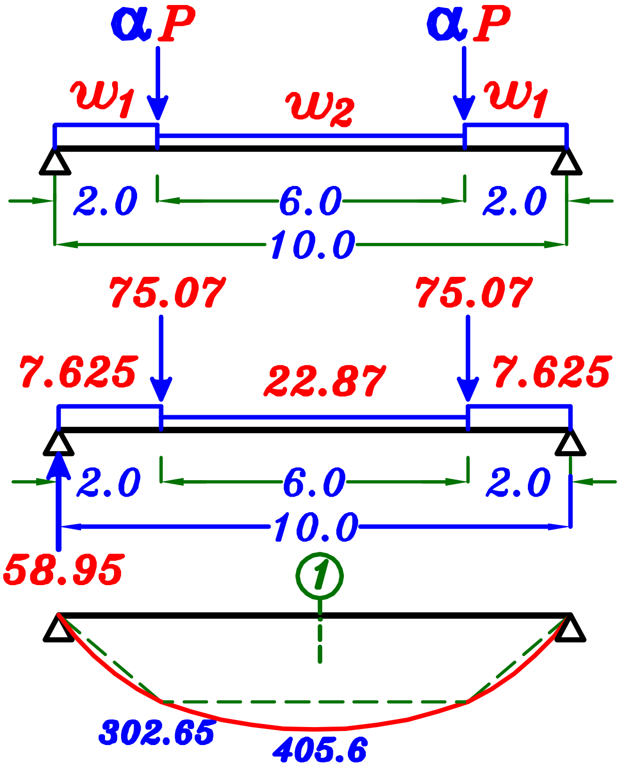
$$w_1 = w_{av.} * \alpha_1 * \alpha$$

$$= 15.25 * 1.0 * 0.50 = 7.625 \ kN\backslash m`$$

$$w_2 = w_{av.} * \alpha_2 * \alpha$$

$$= 15.25 * 3.0 * 0.50 = 22.87 \ kN\backslash m`$$

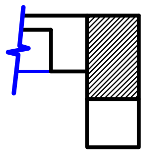


$$\theta_{B_2} = \frac{2.0}{5.0} * 90^\circ = 36^\circ$$

$$M_1 = 405.6 * \frac{\sin 36^\circ}{\sin 90^\circ} = 238.4 \ kN.m$$

$$d = 650 - 50 = 600 \ mm \qquad b = 250 \ mm$$

Sec. 1 $M_{U.L.} = 238.4$ kN.m **R-Sec.**



$$600 = C_1 \sqrt{\frac{238.4 * 10^6}{25 * 250}} \longrightarrow C_1 = 3.07 \longrightarrow J = 0.75$$

$$A_s = \frac{238.4 * 10^6}{0.75 * 420 * 600} = 1261.37 \ mm^2$$

Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 1261.37 \ mm^2$

$$\mu_{min.} \, b \, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b \, d = \left(0.225 * \frac{\sqrt{25}}{420}\right) 250 * 600 = 401.8 \ mm^2$$

$$\therefore A_{s_{req.}} > \mu_{min.} b \, d \quad \therefore \text{Take } A_s = A_{s_{req.}} = 1261.37 \ mm^2$$

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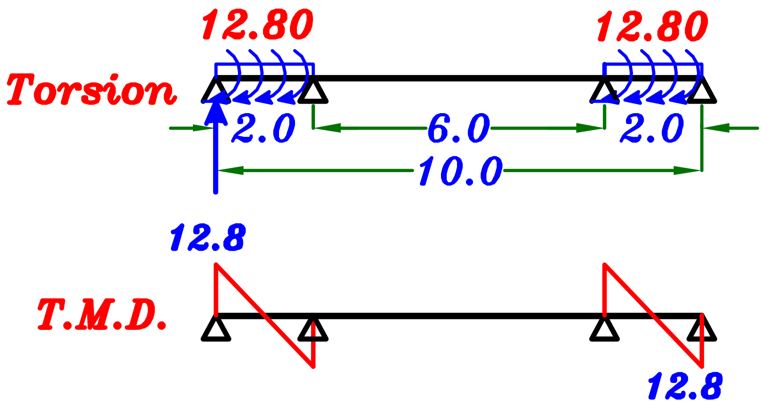
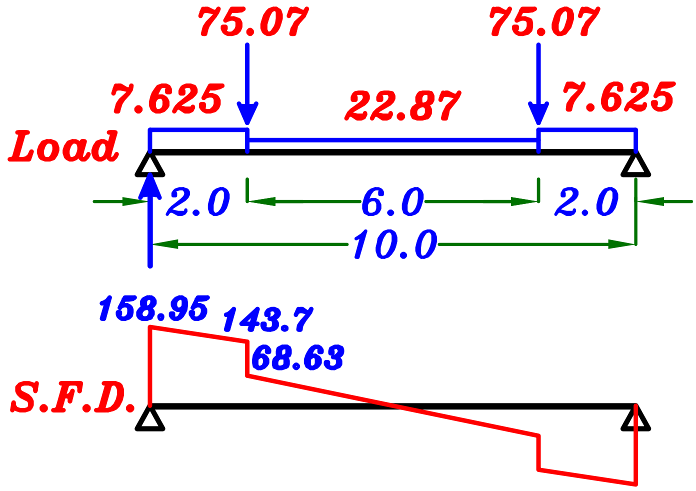
Torsion on Beams.





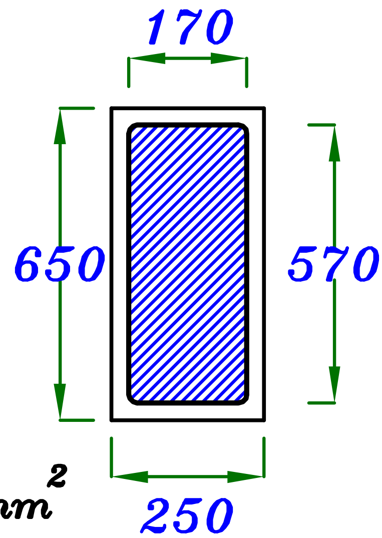
$$q_u = \frac{Q}{b\,d} = \frac{158.95 * 10^3}{250 * 600} = 1.06 \ N/mm^2$$

$$A_{oh} = 170 * 570 = 96900 \ mm^2$$

$$A_o = 0.85 * A_{oh} = 0.85 * 96900 = 82365 \ mm^2$$

$$P_h = 2 * 170 + 2 * 570 = 1480 \ mm$$

$$t_e = \frac{A_{oh}}{P_h} = \frac{96900}{1480} = 65.47 \ mm$$



$$q_{tu} = \frac{M_{tu}}{2\,A_o\,t_e} = \frac{25.6 * 10^6}{2 * 82365 * 65.47} = 2.37 \ N/mm^2$$

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.5}} = 0.98 \ N/mm^2$$

$$q_{t\,min} = (0.06)\sqrt{\frac{25}{1.5}} = 0.245 \ N/mm^2$$

$$q_{u\,max} = (0.7)\sqrt{\frac{25}{1.5}} = 2.85 \ N/mm^2$$

$$\sqrt{q_u^2 + q_{tu}^2} = \sqrt{1.06^2 + 2.37^2} = 2.60 \ N/mm^2 < q_{u\,max} \ \therefore \ o.k.$$

$$\boxed{q_u > q_{cu} \ , \ q_{tu} > q_{t\,min}}$$ $\therefore$ **Use RFT. For Shear + Torsion**

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Check IF stirrups $5\phi 10 \backslash m$ 2 b

are enough to resist Shear + Torsion

ⓐ Torsion.

$x_1 = 220$ mm , $y_1 = 620$ mm , $A_{oh} = 220 * 620 = 136400\ mm^2$

$$A_{str} = \frac{M_{tu}\ S_t}{(1.7)\ A_{oh}\ \left(\frac{F_y}{\gamma_s}\right)} \quad \therefore A_{str} = \frac{(25.6 * 10^6)\ S_t}{(1.7)(96900)(420/1.15)}$$

$$\therefore \boxed{A_{str} = 0.425\ S_t}$$

ⓑ Shear.

$$q_u - \frac{q_{cu}}{2} = \frac{n\ A_s (F_y \backslash \gamma_s)}{b\ \ S_s} \quad \therefore 1.06 - \frac{0.98}{2} = \frac{n\ A_s\ (420/1.15)}{(250)\ S_s}$$

By using $5\phi 10 \backslash m$ 2 b $\quad \therefore \boxed{A_s = 0.397\ \frac{S_s}{n}}$

$n = 2$, $S_t = S_s = \frac{1000}{5.0} = 200$ mm

$\therefore\ A_{str} = 0.425\ S_t = 0.425 * 200 = 85.0\ mm^2$

$A_s = 0.397\ \frac{S_s}{n} = 0.397 * \frac{200}{2.0} = 39.7\ mm^2$

$A_{s_{outer}} = A_{str} + A_s = 85.0 + 39.7 = 124.7 > \phi 10 = 78.5\ mm^2$

Take RFT. of the slab. $8\phi 10 \backslash m$

$n = 2$, $S_t = S_s = \frac{1000}{8.0} = 125$ mm

$\therefore\ A_{str} = 0.425\ S_t = 0.425 * 125 = 53.1\ mm^2$

$A_s = 0.397\ \frac{S_s}{n} = 0.397 * \frac{125}{2.0} = 24.8\ mm^2$

$A_{s_{outer}} = A_{str} + A_s = 53.1 + 24.8 = 77.9 < \phi 10 = 78.5\ mm^2$

$\therefore$ o.k.

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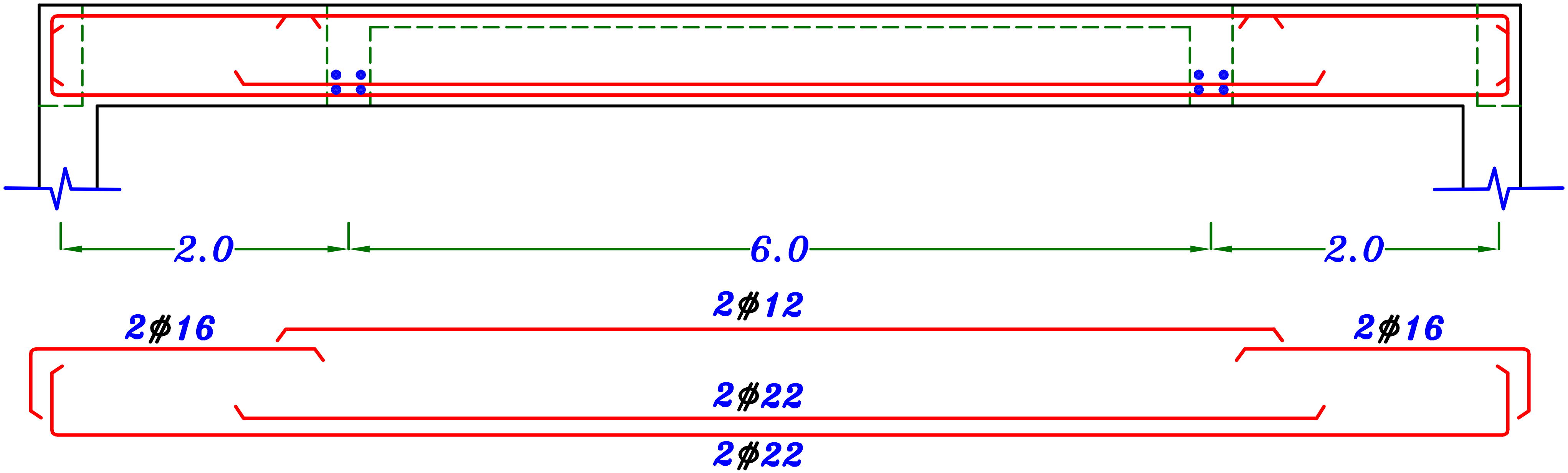
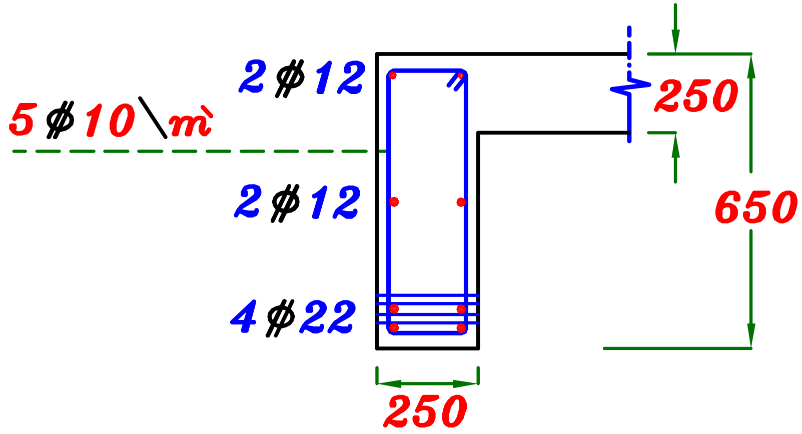
Longitudinal Bars.

$$A_{sl} = \frac{A_{str} * P_h}{S_t}\left(\frac{F_{y\,str.}}{F_{y\,L.b.}}\right) = \frac{(78.5 * 1480)}{125}\left(\frac{420}{420}\right) \quad \therefore A_{sl} = 929.4 \ mm^2$$

$$A_S = A_{S_{beam}} + \frac{A_{sl}}{4} = 1261.37 + \frac{929.4}{4} = 1493.72 \ mm^2 \qquad 4\phi22$$

4- Draw Details of reinforcement of one of the paneled beams.





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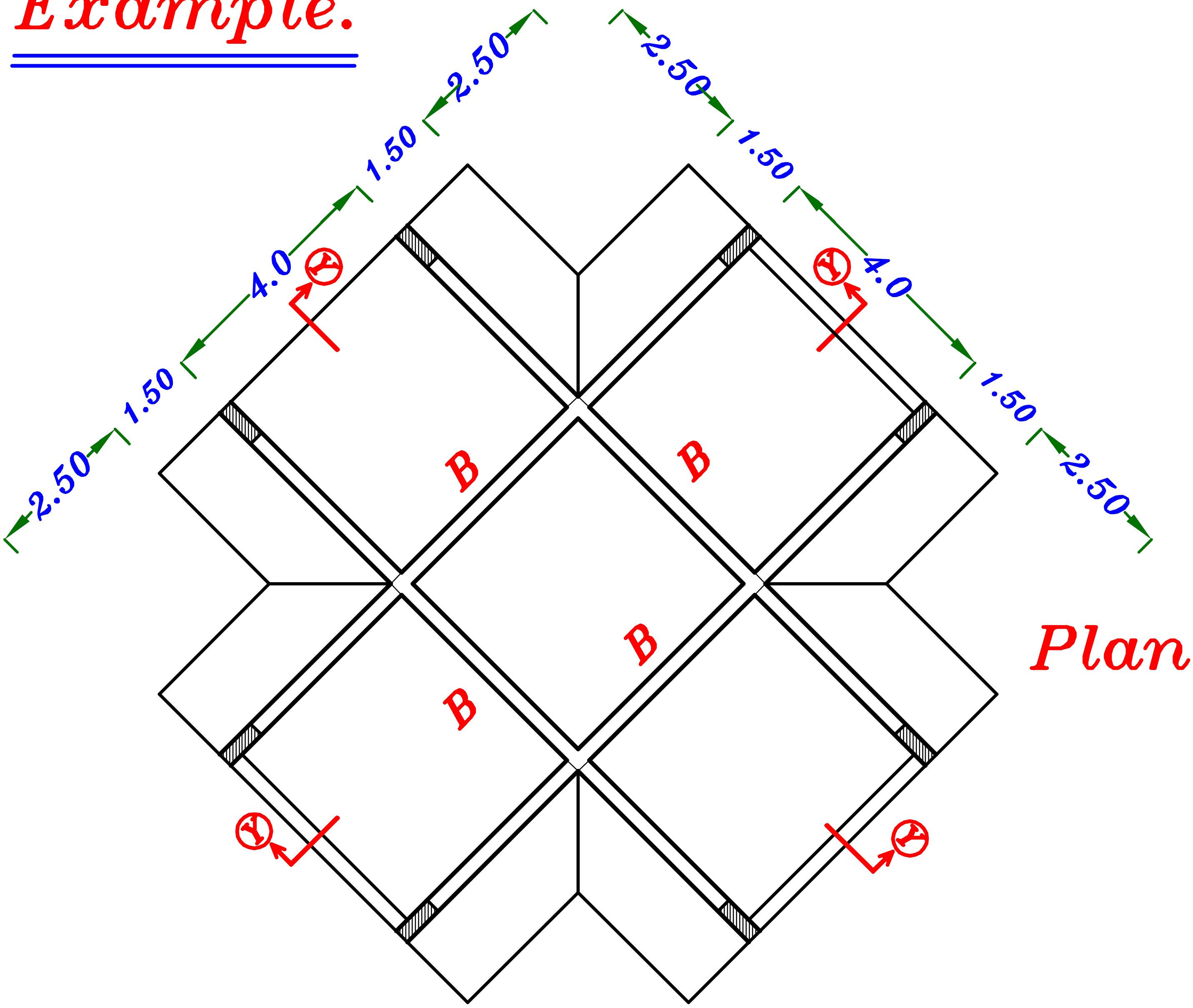
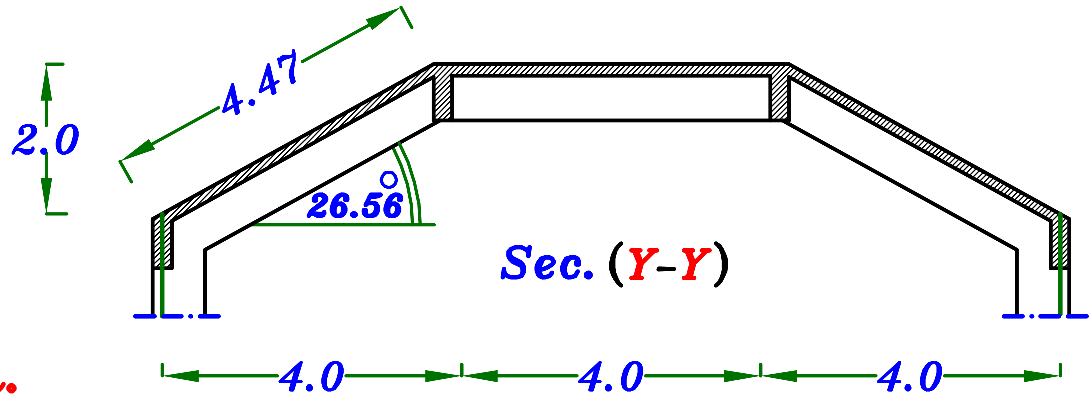
Example.





Data.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. + L.L. = 2.5\ kN\backslash m^2$ Horizontal Projection

Req.

1. Design all Slabs & Draw RFT. in plan.
2. Design the panelled Beams & Draw RFT. in elevation.

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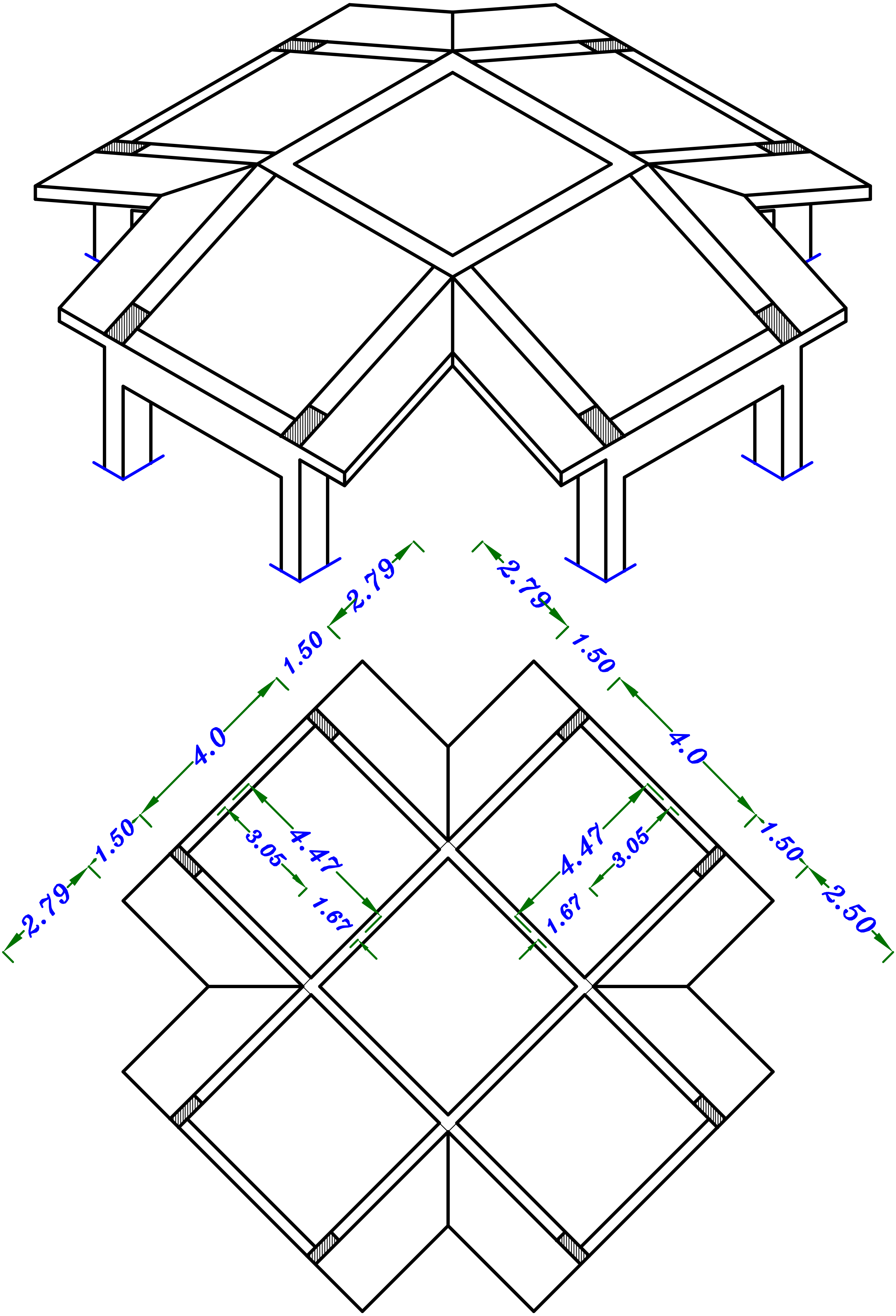

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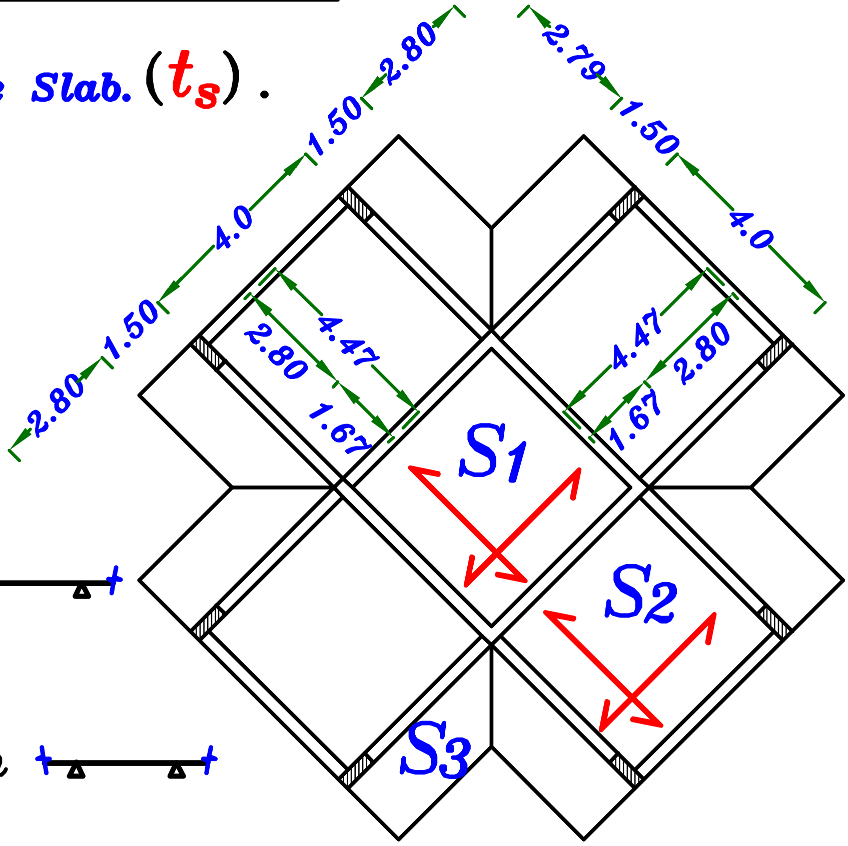
① Design the Slabs as Solid Slabs.

a- Choose the Thickness of the Slab. (t_s).



S_1 two way $L_s = 4.0\ m$

$$t_s = \frac{4000}{45} = 88.9\ mm$$

S_2 two way $L_s = 4.0\ m$

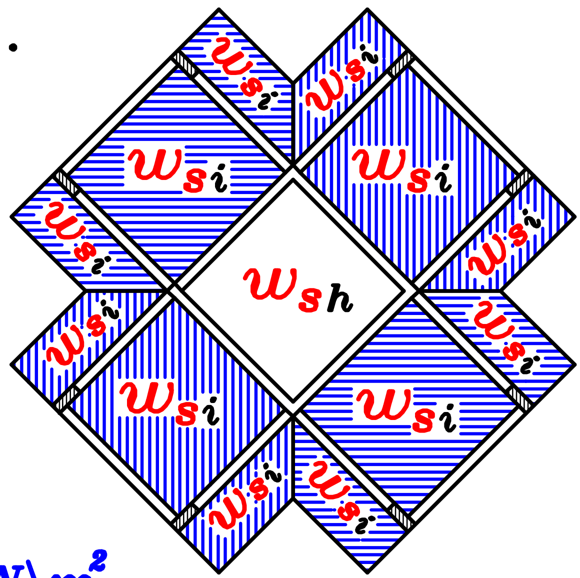
$$t_s = \frac{4000}{45} = 88.9\ mm$$

S_3 Cantilever $L_c = 1.5\ m$

$$t_s = \frac{1500}{10} = 150\ mm$$

Take (t_s) the bigger value $\boxed{t_s = 150\ mm}$

b- Get the Loads on the Slab (w_s).



$$w_{sh} = 1.5\,(0.15 * 25 + 2.5) = 9.37\ kN\backslash m^2$$

$$w_{si} = 1.5\,(0.15 * 25 + 2.5 * \cos 26.56^\circ) = 8.98\ kN\backslash m^2$$

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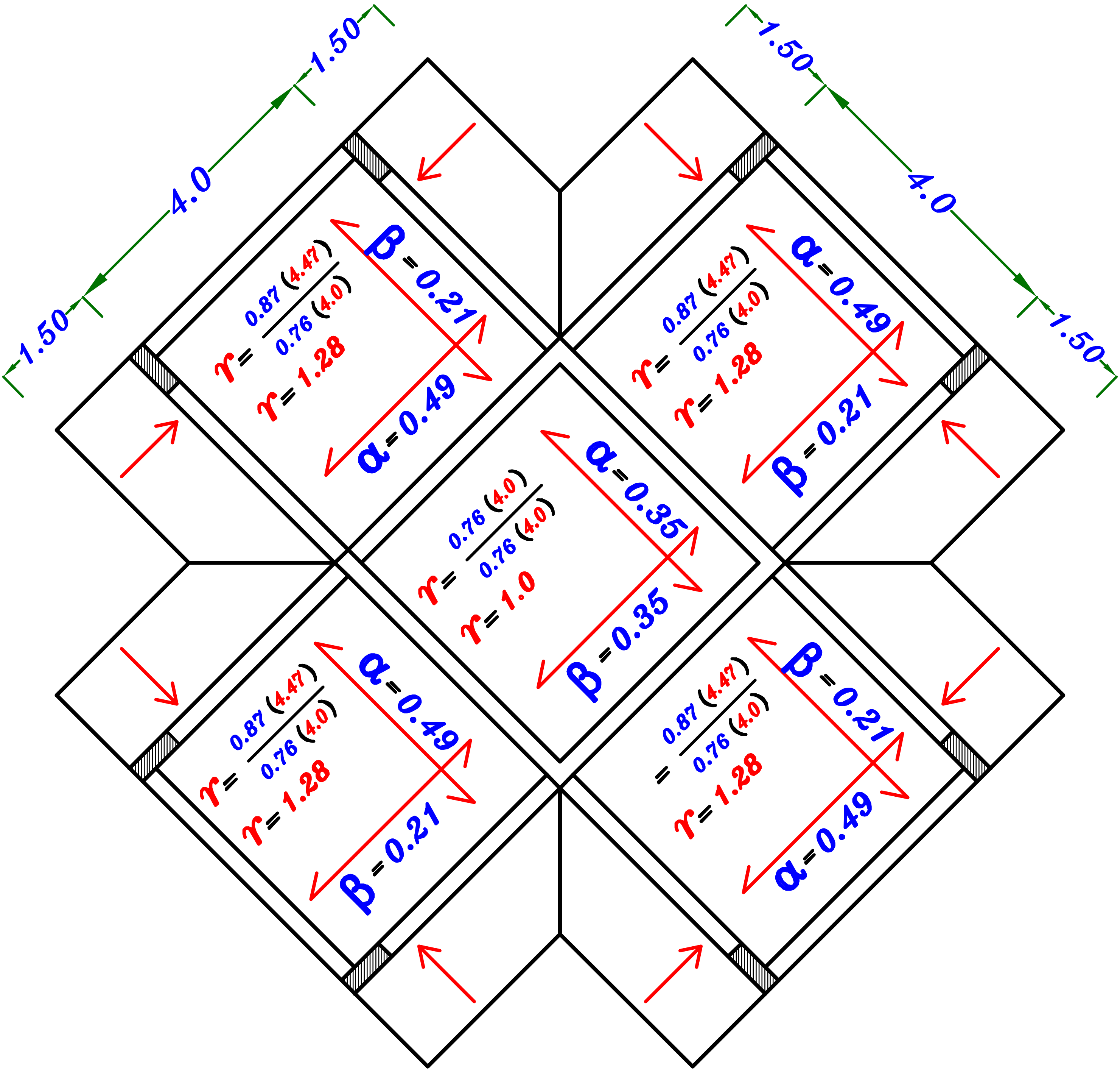
c - Get the Load Factors α, β



$$\boxed{\alpha = 0.5\,\gamma - 0.15}$$

$$\boxed{\beta = \frac{0.35}{\gamma^2}}$$

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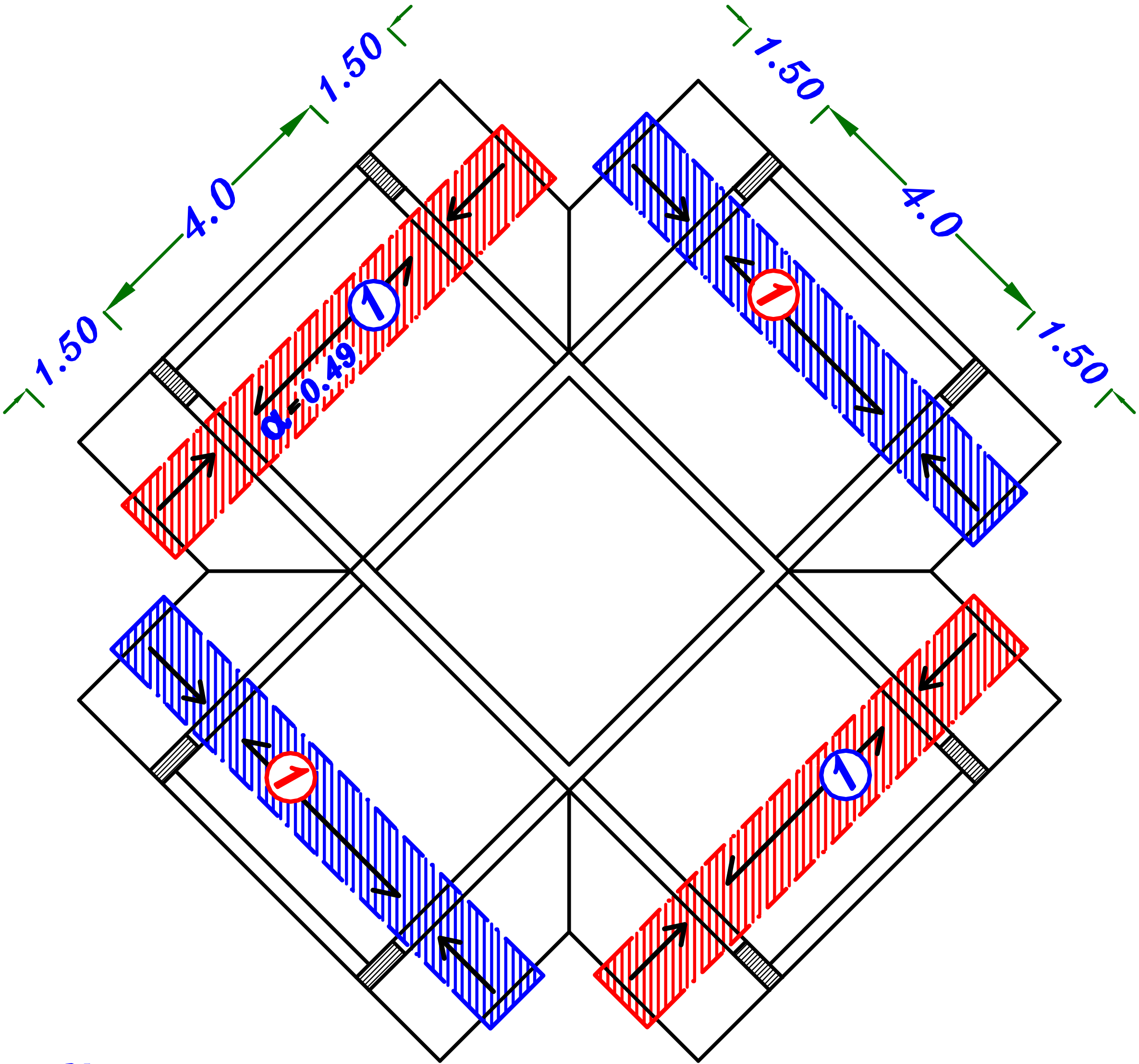
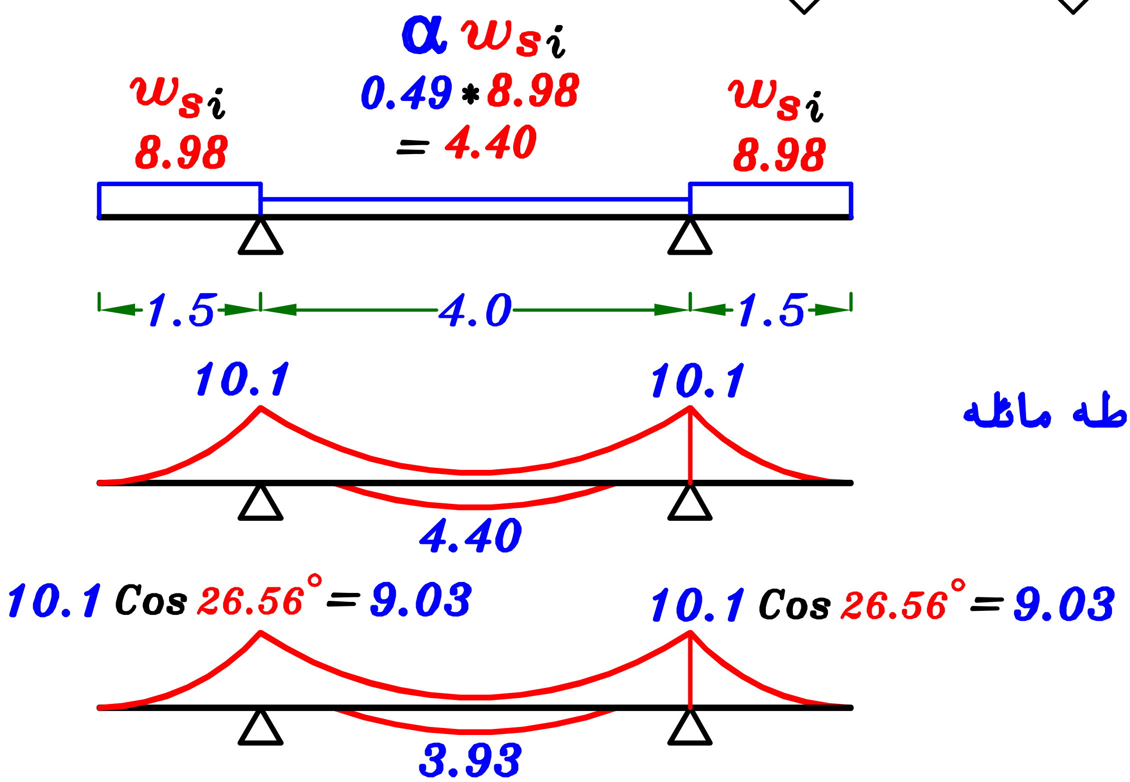
d- Take a strips in the slab (at the Load direction)
And then Get (B.M.) on the Slab & Design the slab.

Strip ①





شريحه أفقيه فى بلاطه مائله

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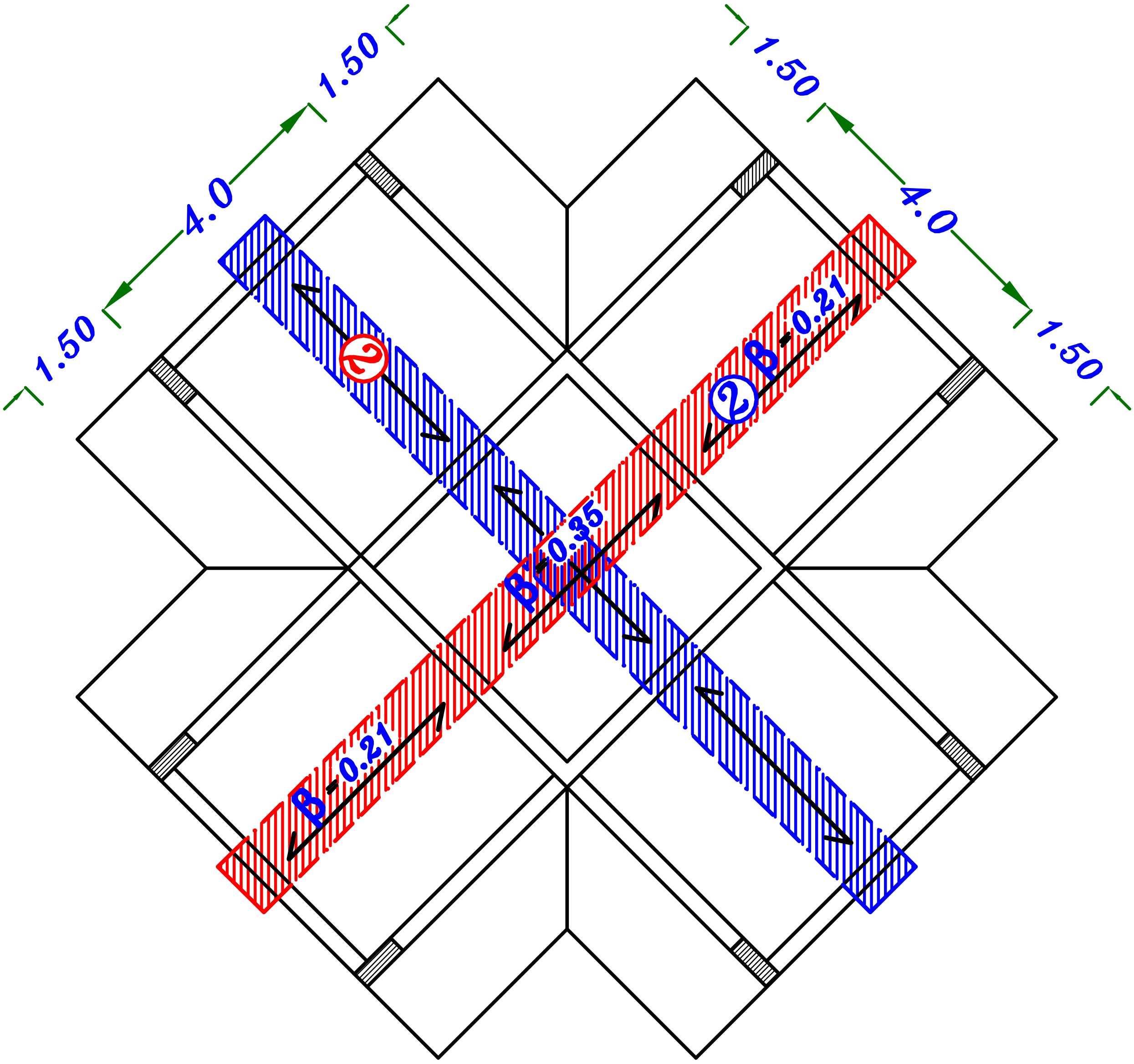
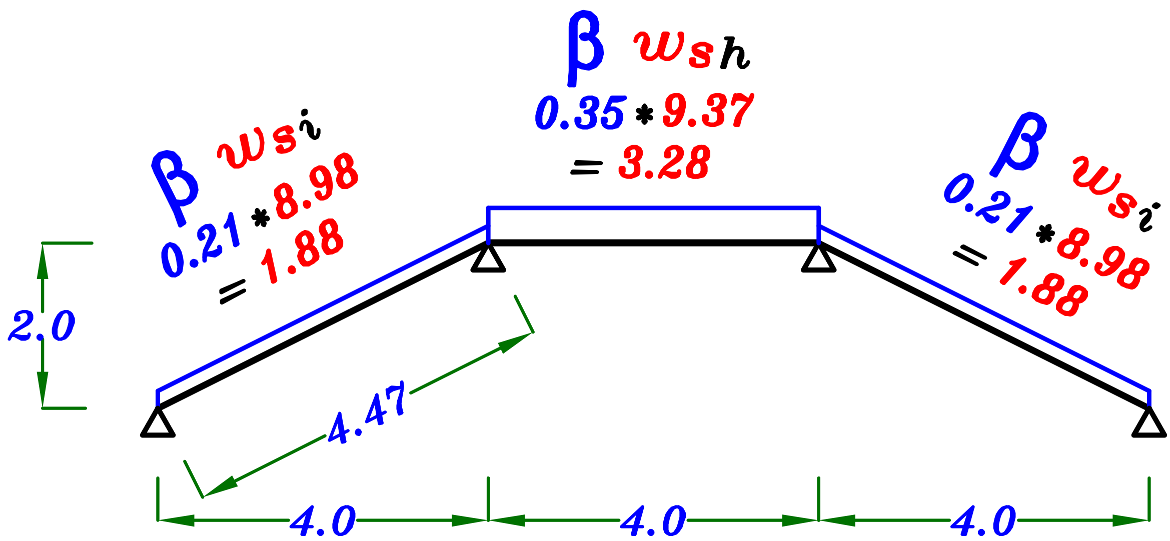
d- Take a strips in the slab (at the Load direction)
And then Get (B.M.) on the Slab & Design the slab.

Strip 1





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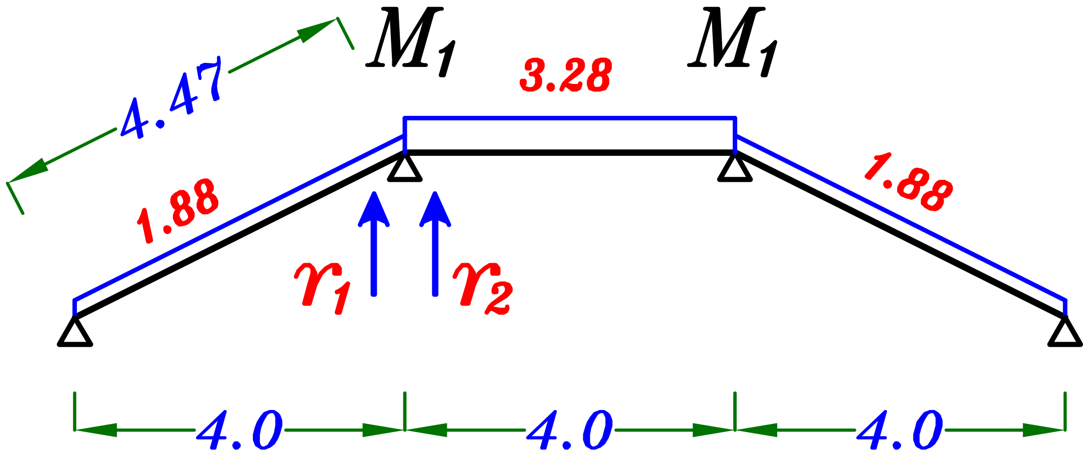
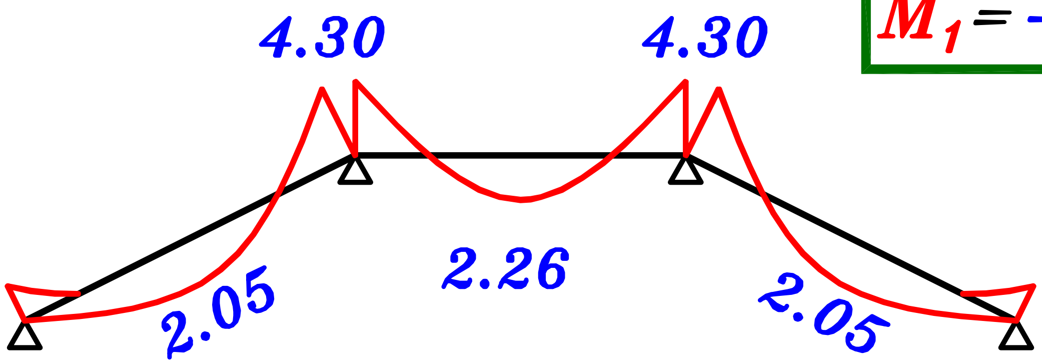


$$r_1 = \frac{wLL'^2}{24} = \frac{1.88 * 4.0 * 4.47^2}{24} = 6.26 \quad , \quad r_2 = \frac{wL^3}{24} = \frac{3.28 * 4.0^3}{24} = 8.74$$

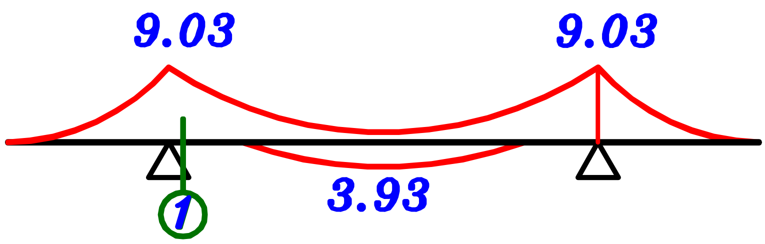
Equation of M_1

$$0.0 + 2\,M_1\,(4.47 + 4.0\,) + M_1\,(4.0) = -6\,(6.26 + 8.74\,)$$

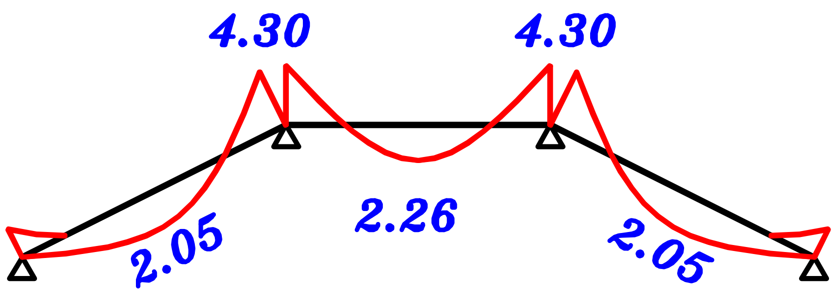
$$\boxed{M_1 = -4.30 \text{ kN.m.}}$$



Strip 1



Strip 2



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Sec. ① $M_{U.L.} = 9.03$ kN.m\m

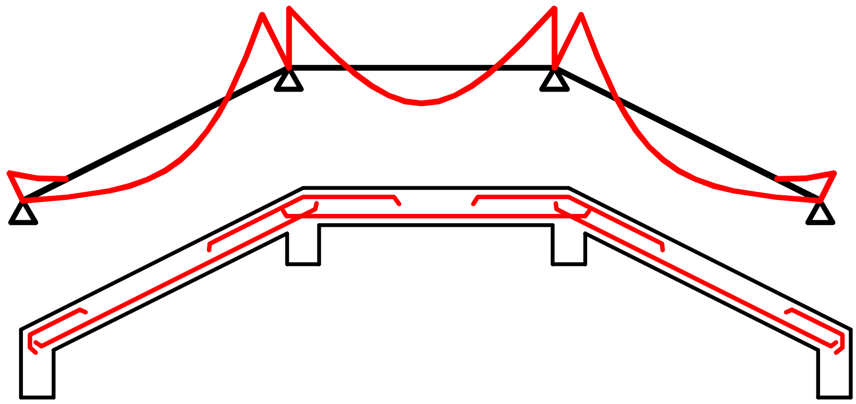
$t_s = 150$ mm , $d = 150 - 20 = 130$ mm , $B = 1000$ mm عرض الشريحه

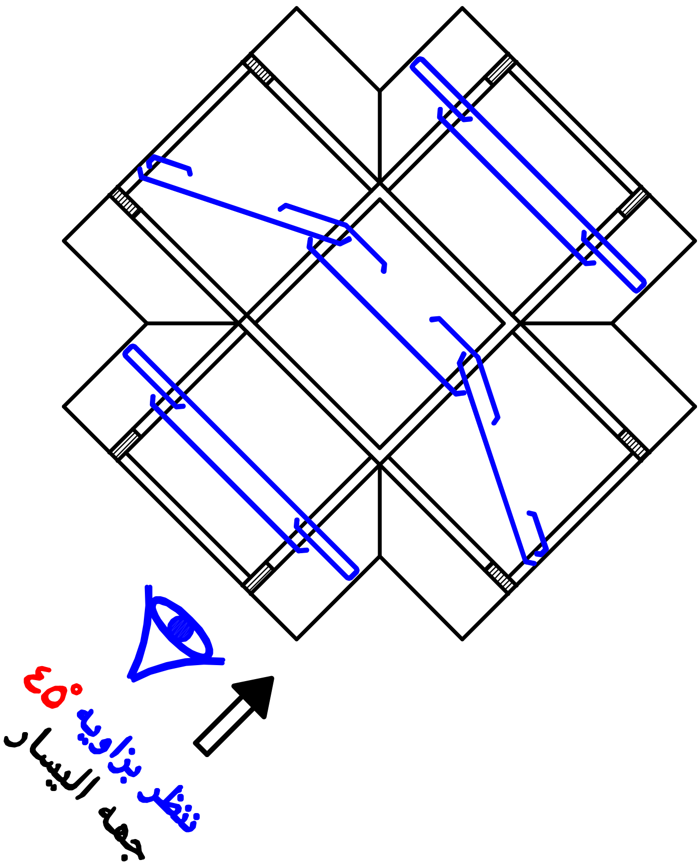
$$130 = C_1\sqrt{\frac{9.03 * 10^6}{25 * 1000}} \longrightarrow C_1 = 6.84 \longrightarrow J = 0.826$$

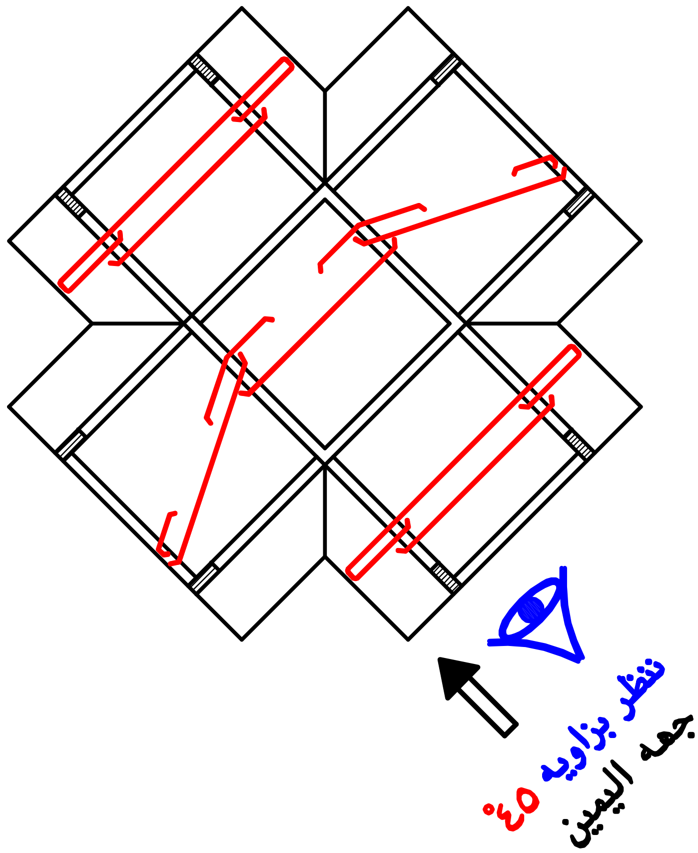
$$A_s = \frac{9.03 * 10^6}{0.826 * 360 * 130} = 233.6 \ mm^2/m` \qquad 5\phi10 \backslash m`$$

∴ سيؤخذ تسليح باقى القطاعات $5\phi10 \backslash m`$

Details of RFT. For the Slab.







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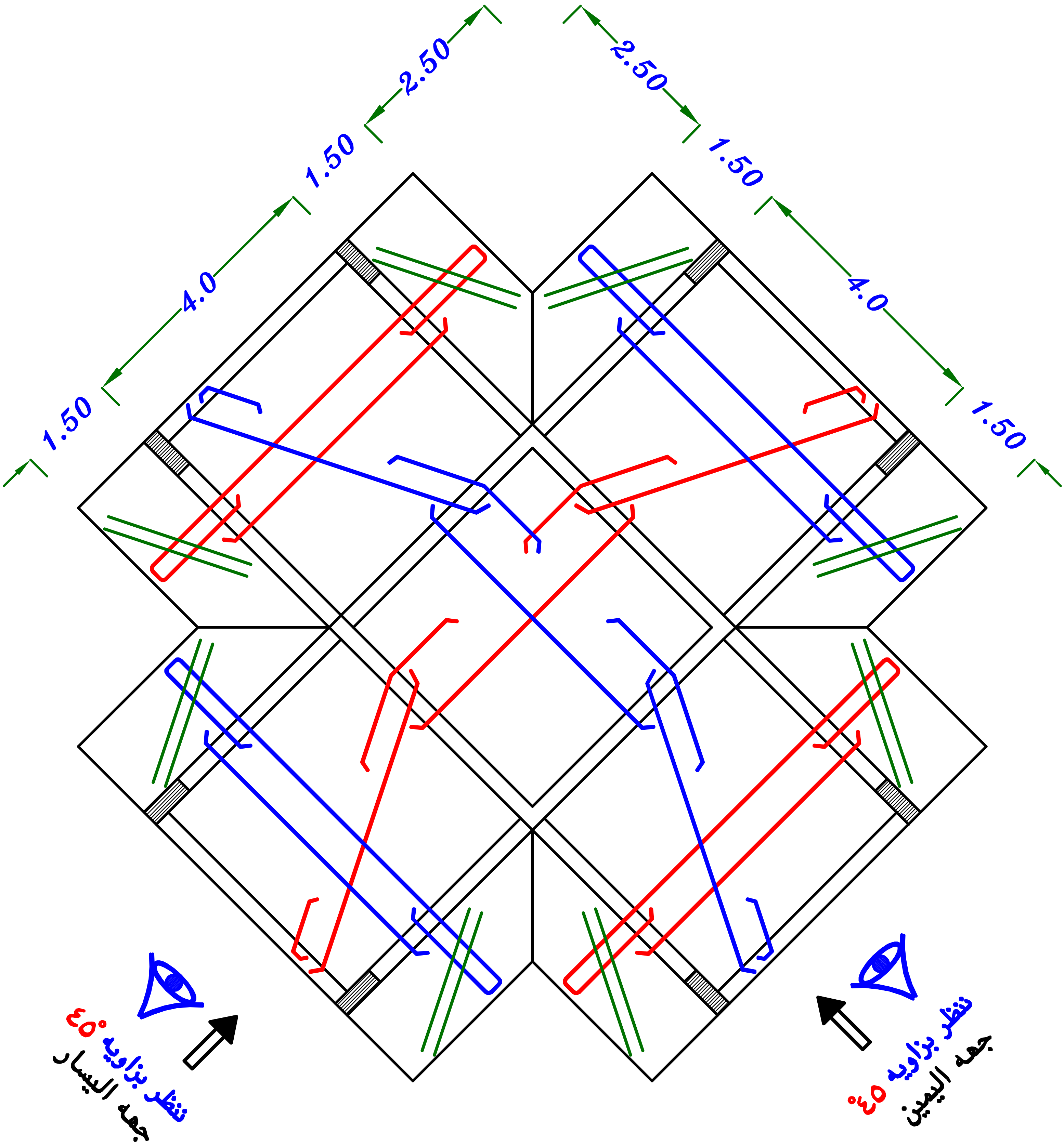
Details of RFT. For the Slab.



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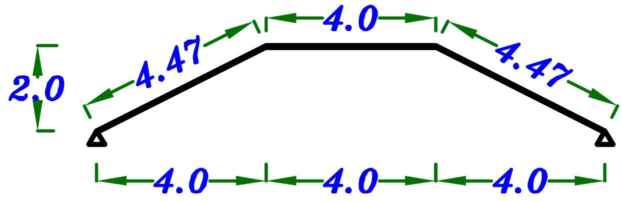
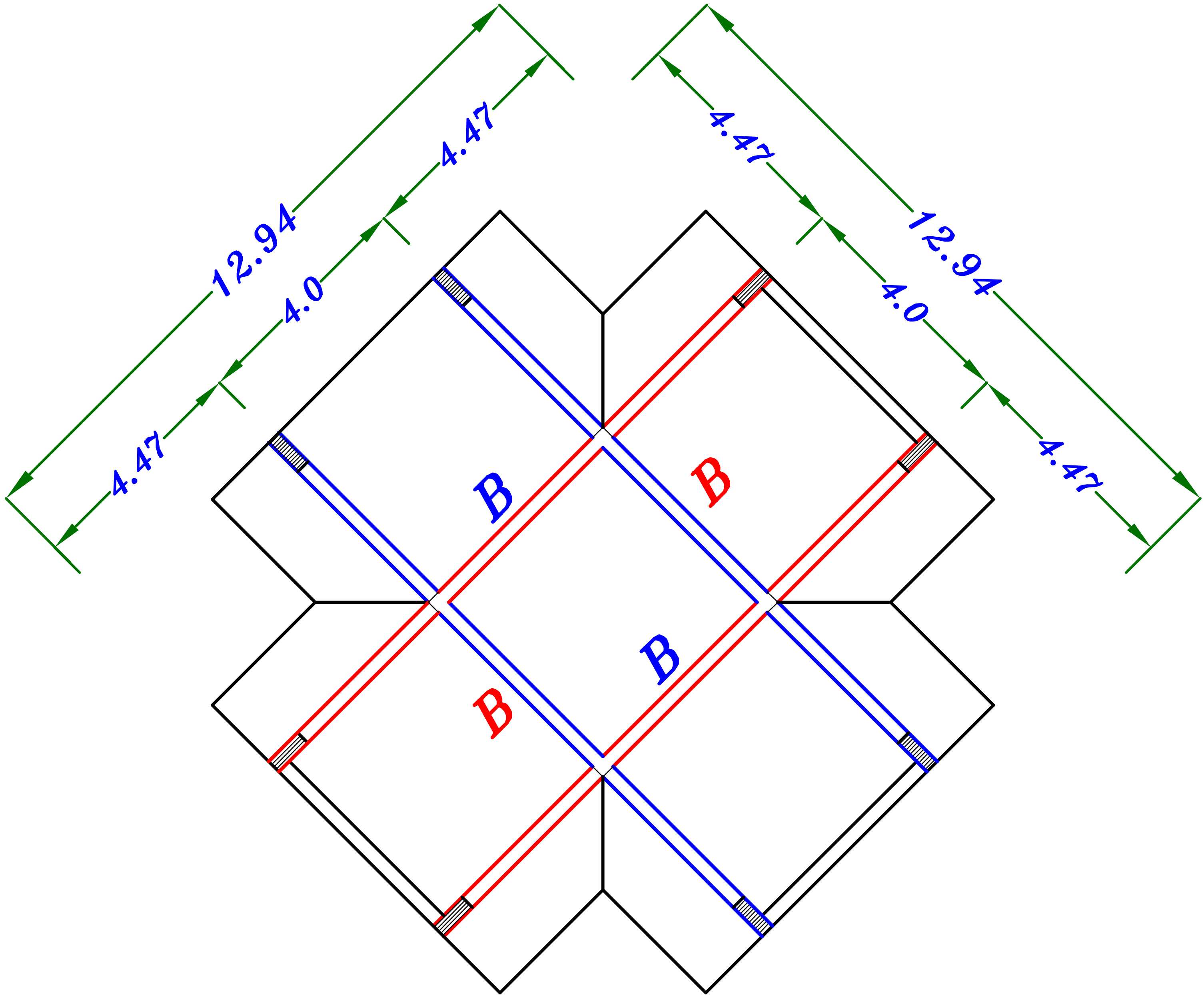
② Design of Panelled Beams.

$$L' = 4.47 + 4.0 + 4.47 = 12.94\ m$$





a – Get the Dimensions of the beam. (b , t)

Take $b = 0.25\ m$

$$t = \frac{L_s}{16} = \frac{12940}{16} = 0.808\ m$$

$\boxed{b = 0.25\ m}$ $\boxed{t = 0.85\ m}$

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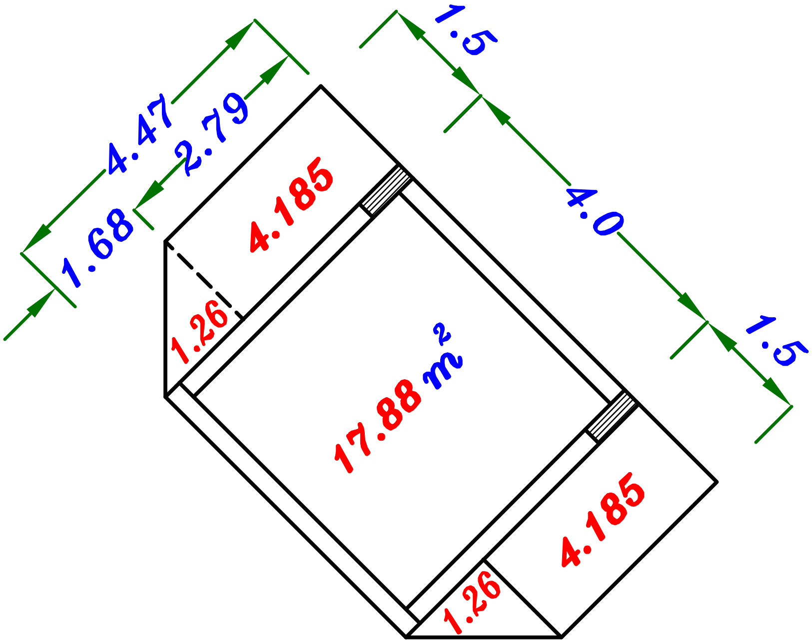
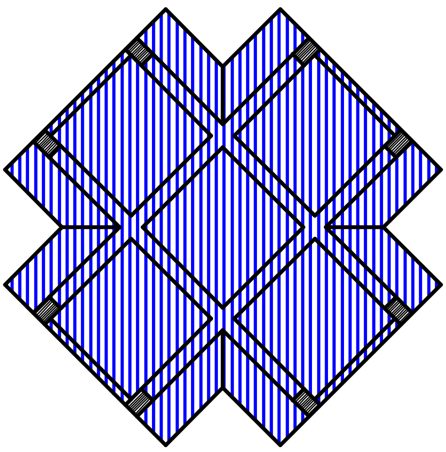
b – Get the Loads on the Slab. (w_{av})



Inclined area

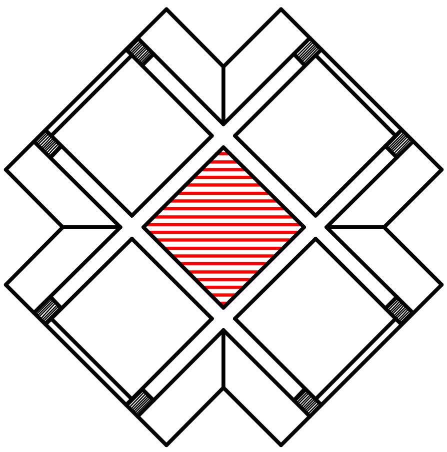
$= 4\left[2 * 1.26 + 2 * 4.185 + 17.88\right]$

$= 115.08 \ m^2$



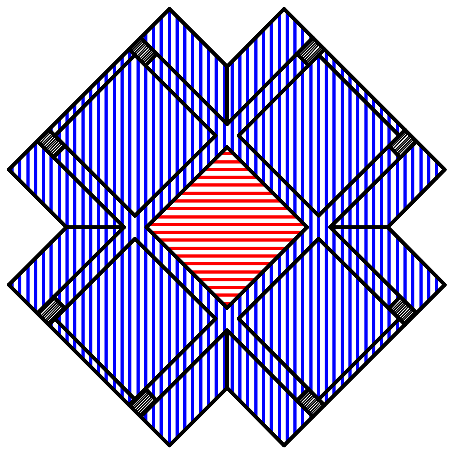
Inclined area

$= 115.08 \ m^2$



Horizontal area

$= 16.0 \ m^2$



Total area

$= 131.08 \ m^2$

$$w_{av.} = \frac{\sum Weight}{Area}$$

$$w_{av.} = \frac{w_{sh} * HL. \ area + w_{si} * Inclined \ area + Panelled \ Beams}{Total \ area}$$

$$w_{av.} = \frac{9.37 * 16.0 + 8.98 * 115.08 + 1.4\left(0.25\left(0.85 - 0.15\right)\left[4 * 12.94\right] * 25\right)}{131.08}$$

$$\boxed{w_{av.} = 11.44 \ kN\backslash m^2}$$

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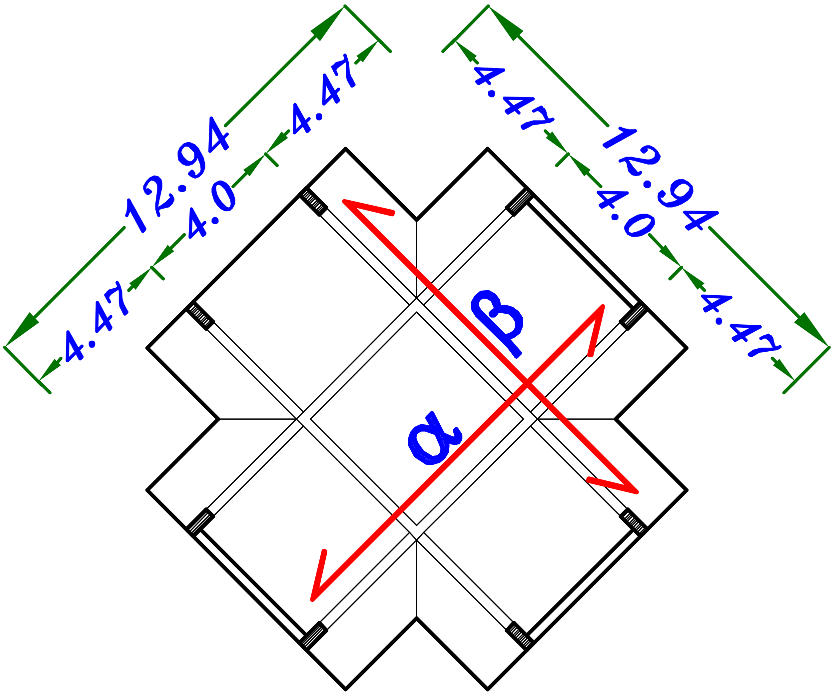
c - Calculate α, β By using Grashoff.

$$r = \frac{m\ L}{m`\ L_s} = \frac{(1.0)\ 12.94}{(1.0)\ 12.94} = 1.0$$

$$\alpha = \frac{r^4}{1+r^4} = \frac{(1.0)^4}{1+(1.0)^4} = 0.50$$

$$\beta = \frac{1}{1+r^4} = \frac{1}{1+(1.0)^4} = 0.50$$

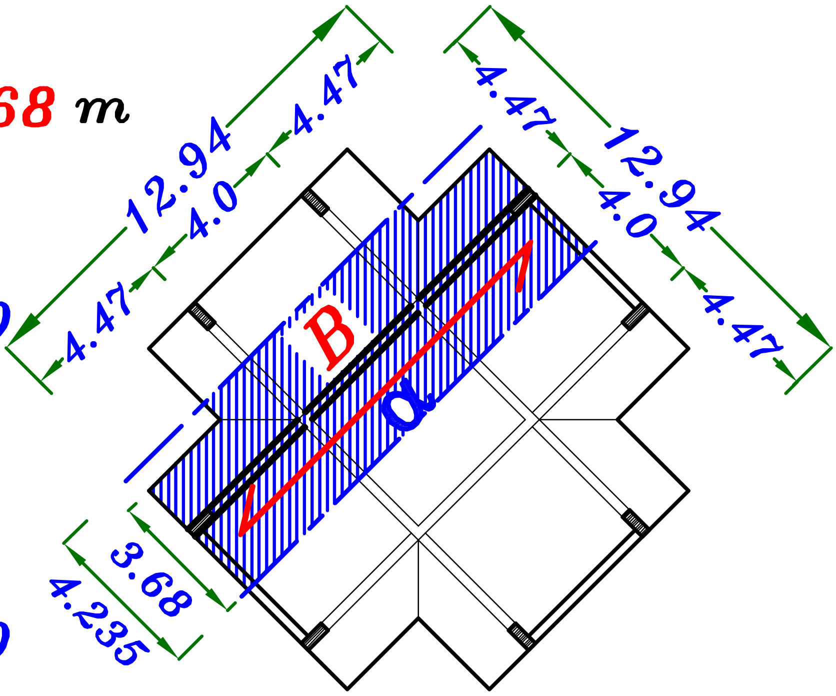


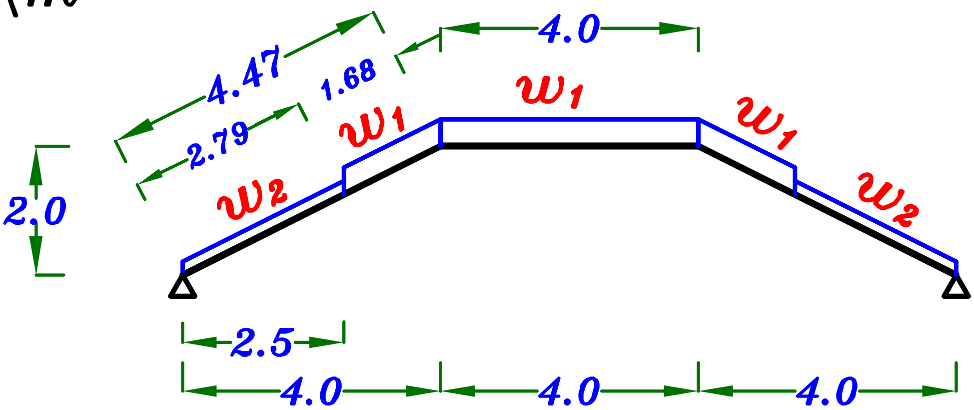
B

$a_1 = 4.235\ m$, $a_2 = 3.68\ m$

$$w_1 = w_{av.} * a_1 * \alpha$$
$$= 11.44 * 4.235 * 0.50$$
$$= 24.22\ kN\backslash m`$$

$$w_2 = w_{av.} * a_2 * \alpha$$
$$= 11.44 * 3.68 * 0.50$$
$$= 21.05\ kN\backslash m`$$





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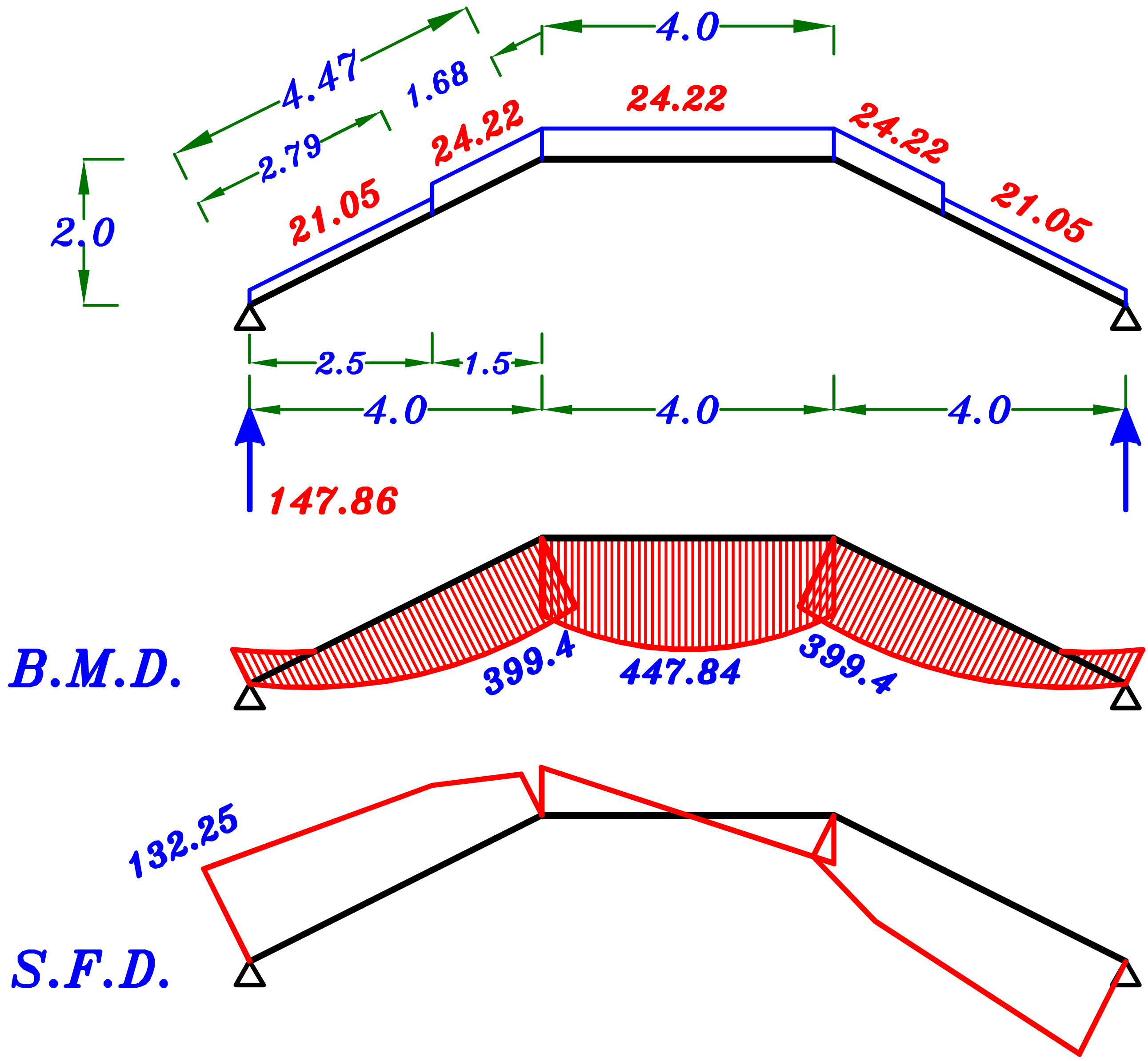


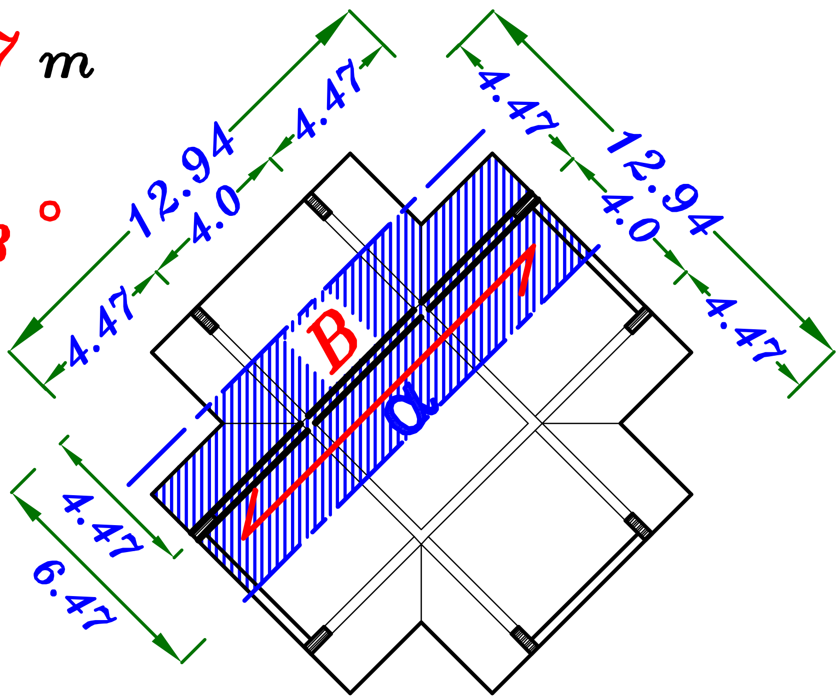
e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 4.47\ m\ ,\quad \frac{L}{2} = 6.47\ m$$

$$\theta_{B_1} = \frac{4.47}{6.47} * 90^\circ = 62.18^\circ$$

$$M_1 = 447.84 * \frac{\sin 62.18^\circ}{\sin 90^\circ}$$

$$= 396.07\ kN.m$$



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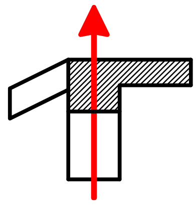
F- Design the Panelled Beam. B

Cover = 70 mm Symmetric

$t = 850\ mm \qquad d = 850 - 70 = 780\ mm$



$$B = \begin{cases} C.L.-C.L. = 2.0\ m = 2000\ mm \\ 6\ t_s + b = 6 * 150 + 250 = 1150\ mm \\ K \frac{L}{10} + b = 1.0 * \frac{12940}{10} + 250 = 1544\ mm \end{cases}$$

$\boxed{B = 1150\ mm}$

$$780 = C_1 \sqrt{\frac{396.07 * 10^6}{25 * 1150}} \longrightarrow C_1 = 6.64 \longrightarrow J = 0.826$$

$$A_s = \frac{396.07 * 10^6}{0.826 * 360 * 780} = 1707.6\ mm^2$$

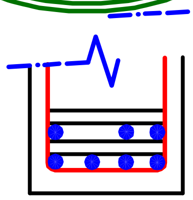
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 1707.6\ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 780 = 609.3\ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\ d \quad \therefore$ Take $A_s = A_{s_{req.}} = 1707.6\ mm^2$ $\quad 7 \phi 18$

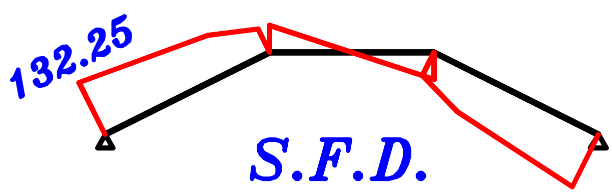
Stirrup Hangers $= (0.1 \rightarrow 0.2)\ A_s = (170 \rightarrow 340\ mm^2)$

$4 \phi 10$



Check Shear.

$$q_{cu} = (0.24) \sqrt{\frac{25}{1.50}} = 0.98\ N\backslash mm^2$$



$$q_{u_{max}} = (0.70) \sqrt{\frac{25}{1.50}} = 2.85\ N\backslash mm^2$$

$$q_s = \frac{Q_{max}}{b\ d} = \frac{132.25 * 10^3}{250 * 780} = 0.67\ N\backslash mm^2 \quad \therefore q_s < q_{cu}$$

$\therefore$ Use min. Shear RFT. $\quad 5 \phi 8 \backslash m$

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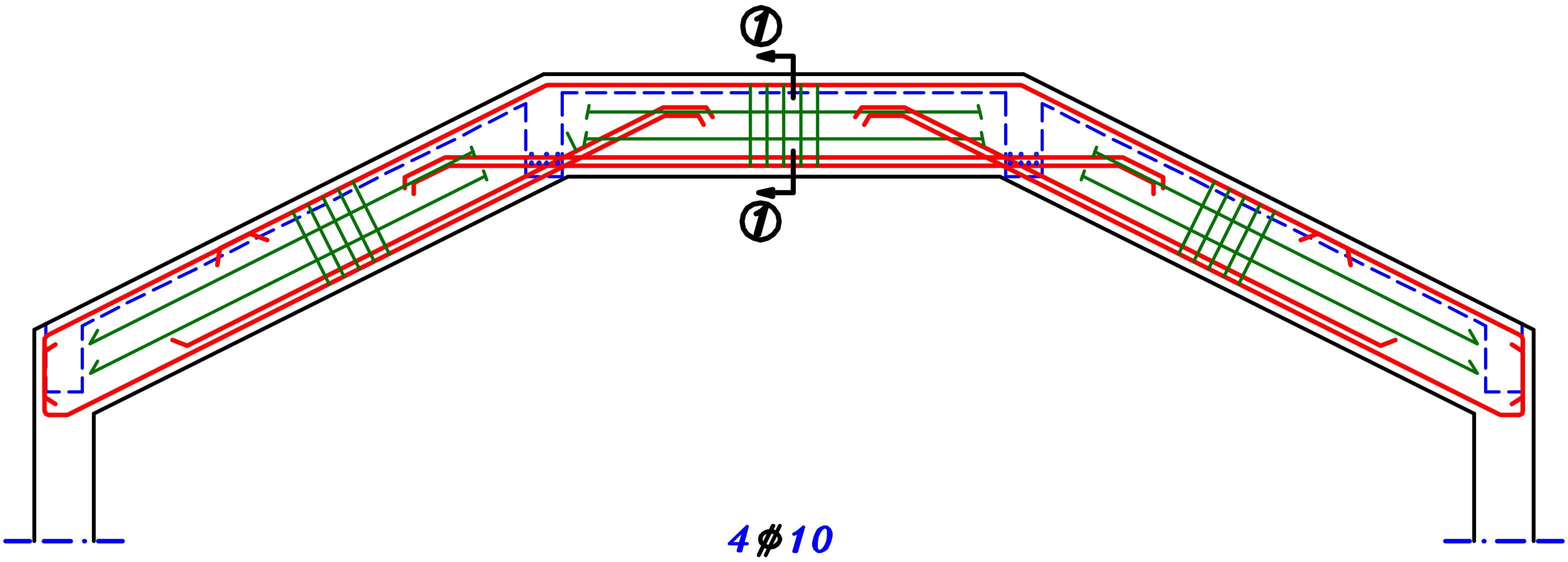
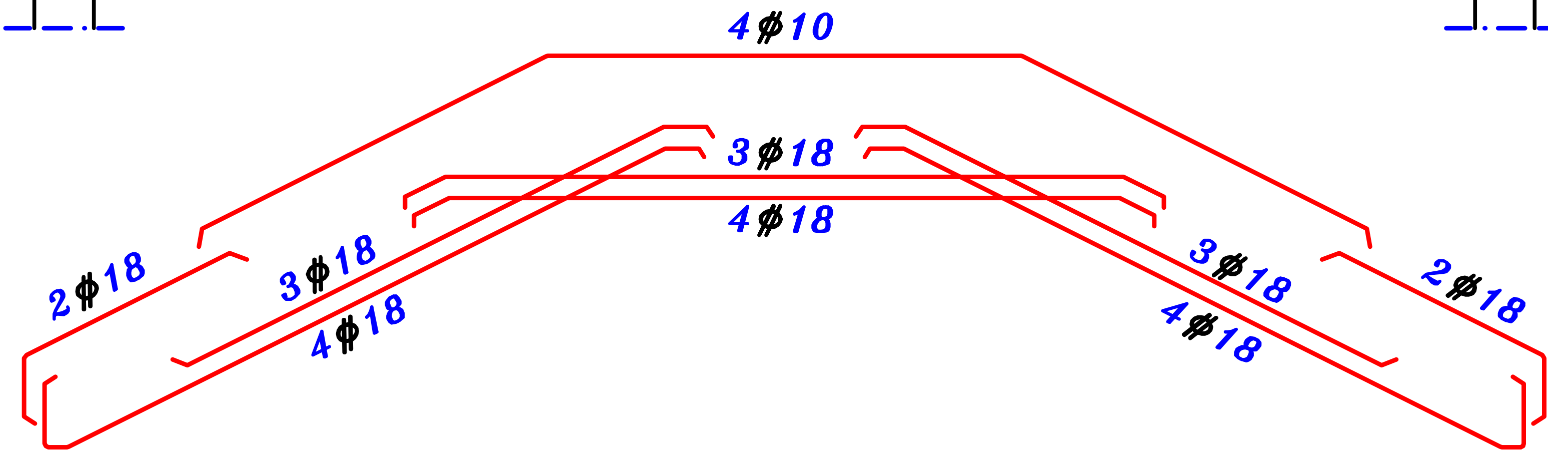
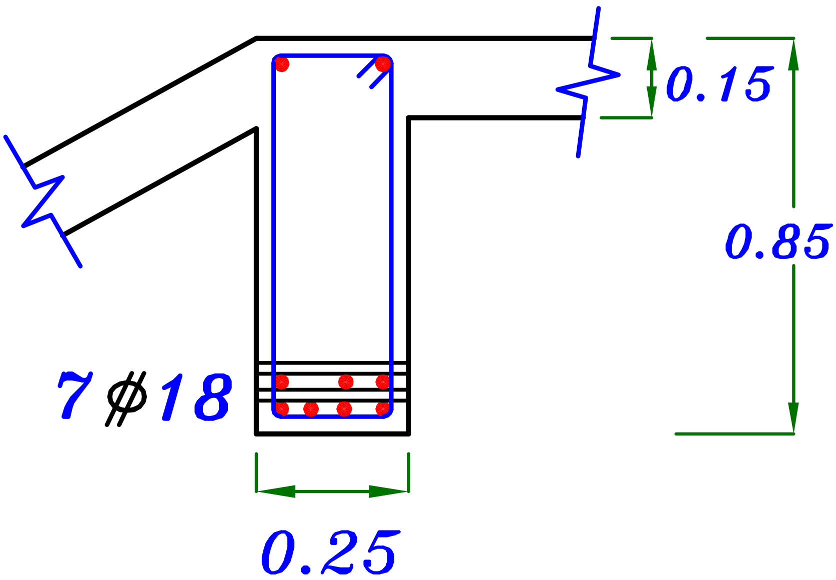
g – Draw Details of RFT. For the Beam.







Sec. (1–1)

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Note.

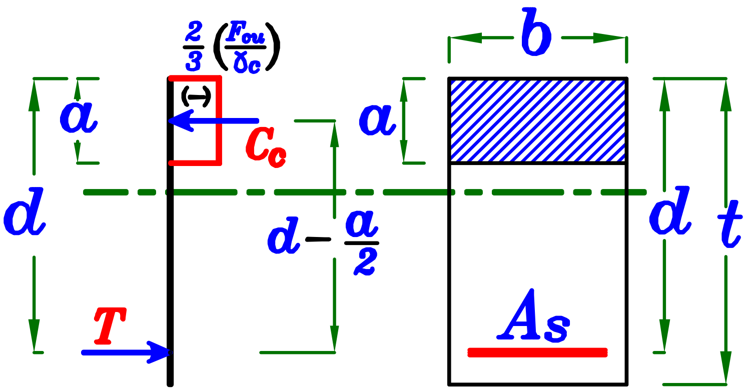
اذا احتجنا حساب الـ moment المصمم عليه القطاع و يسمى $M_{U.L.}$

Calculation of $M_{U.L.}$

For R – Sec.

$$C_c = \frac{2}{3}\frac{F_{cu}}{\gamma_c}\, a\, b$$

$$T = \frac{F_y}{\gamma_s} * A_s$$



From $\frac{2}{3}\frac{F_{cu}}{\gamma_c} * a * b = \frac{F_y}{\gamma_s} * A_s \xrightarrow{get} a$

IF $\boxed{a}$

IF $a \leqslant 0.1\, d$

take $a = 0.1\, d$

$\therefore M_{U.L.} = A_s \frac{F_y}{\gamma_s}\left(d - \frac{a}{2}\right)$

$\therefore M_{U.L.} = A_s \frac{F_y}{\gamma_s}\left(d - \frac{0.1 d}{2}\right)$

$\therefore M_{U.L.} = A_s\, F_y\, d\, \frac{1}{1.15}\left(1 - \frac{0.1}{2}\right)$

$$\therefore \boxed{M_{U.L.} = 0.826\, A_s\, F_y\, d}$$

IF $a > 0.1\, d$

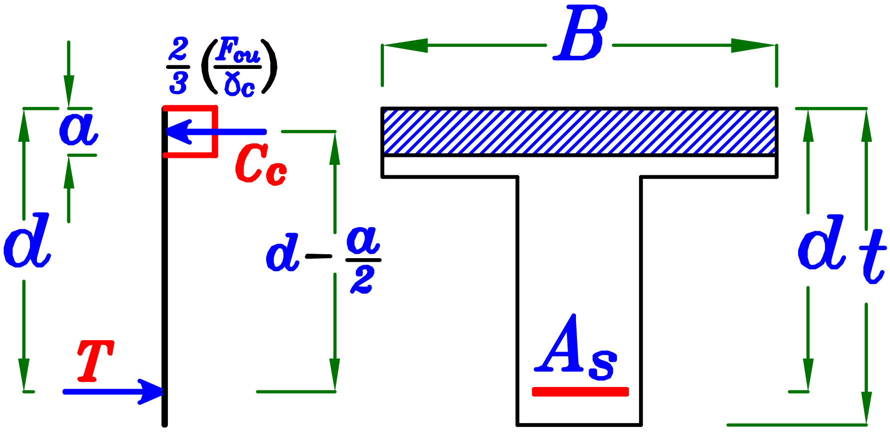
$$\therefore \boxed{\begin{aligned} M_{U.L.} &= \frac{2}{3}\frac{F_{cu}}{\gamma_c}\, a\, b\left(d - \frac{a}{2}\right) \\ &= A_s * \frac{F_y}{\gamma_s}\left(d - \frac{a}{2}\right) \end{aligned}}$$

For T – Sec.

the same as R–sec. but with B

$$C_c = \frac{2}{3}\frac{F_{cu}}{\gamma_c}\, a\, B$$

$$T = \frac{F_y}{\gamma_s} * A_s$$



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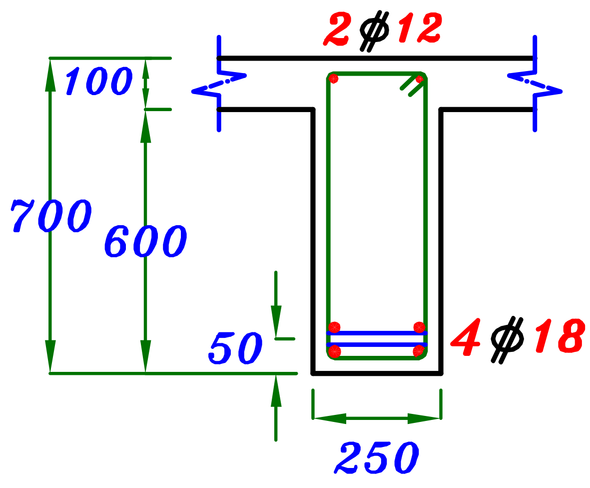
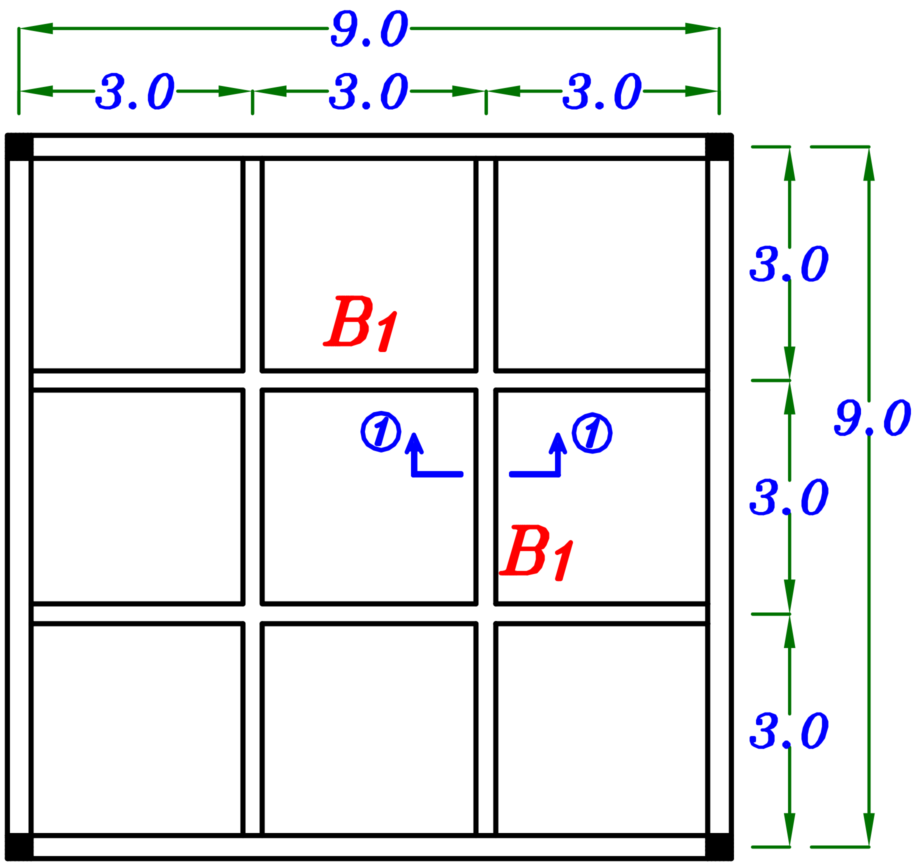
Example.



Sec (1−1)



Data.

$F_{cu} = 25\ N\backslash mm^2$ $\qquad F_y = 360\ N\backslash mm^2$

$F.C. = 1.5\ kN\backslash m^2$ $\qquad L.L. = ??\ kN\backslash m^2$

Req.

Calculate the maximum Live Load Could be applied to the roof.

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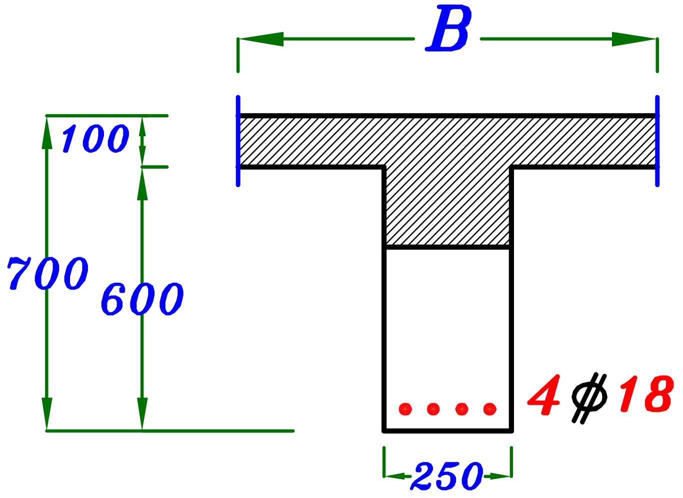
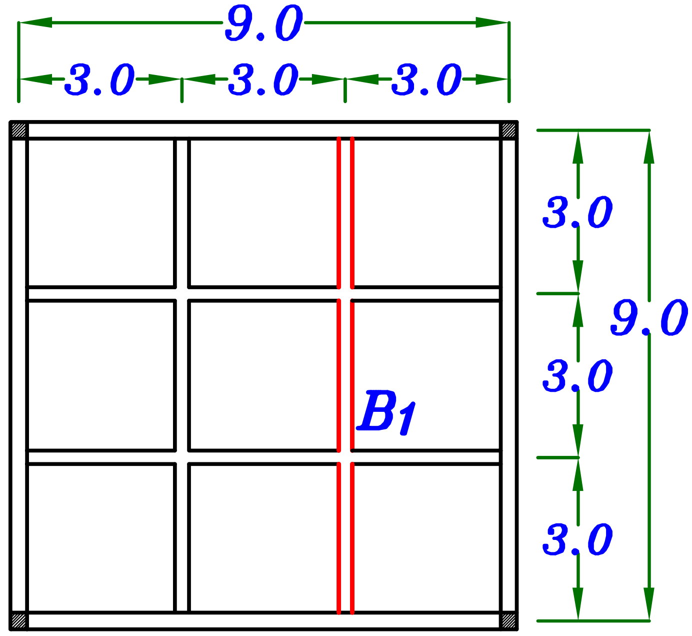




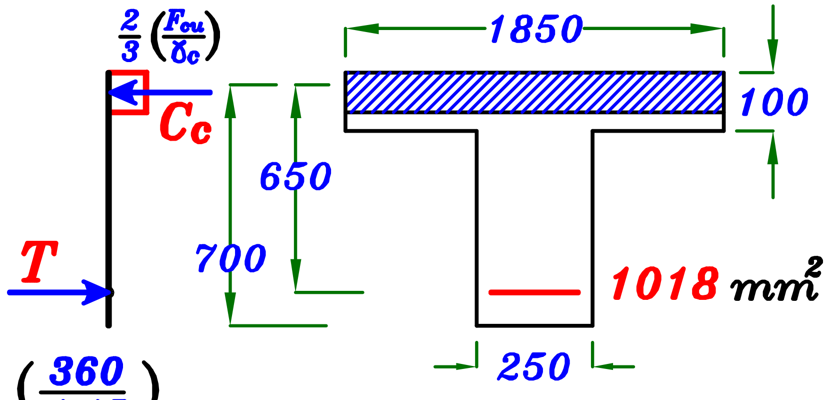
$$A_s = 4\phi 18 = 1018\ mm^2$$

$$B = \begin{cases} C.L.-C.L. = 3.0\ m = 3000\ mm \\ 16\, t_s + b = 16 * 100 + 250 = 1850\ mm \\ K\dfrac{L}{5} + b = 1.0 * \dfrac{9000}{5} + 250 = 2050\ mm \end{cases} \qquad \boxed{B = 1850\ mm}$$

Calculate $M_{U.L.}$ ------ اكبر moment يتحمله هذا القطاع

$$C_c = T$$

$$\frac{2}{3}\frac{F_{cu}}{\gamma_c}\, a\, B = \frac{F_y}{\gamma_s} * A_s$$



$$\frac{2}{3}\left(\frac{25}{1.5}\right)(a)(1850) = (1018)\left(\frac{360}{1.15}\right)$$

$$\therefore a = 15.5\ mm < 0.1\, d \xrightarrow{Take} a = 0.1\, d = 65\ mm$$

$$\therefore M_{U.L.} = 0.826\, A_s\, F_y\, d = 0.826 * 1018 * 360 * 650 = 196763112\ N.mm$$

$$\therefore \boxed{M_{U.L.} = 196.76\ kN.m.}$$

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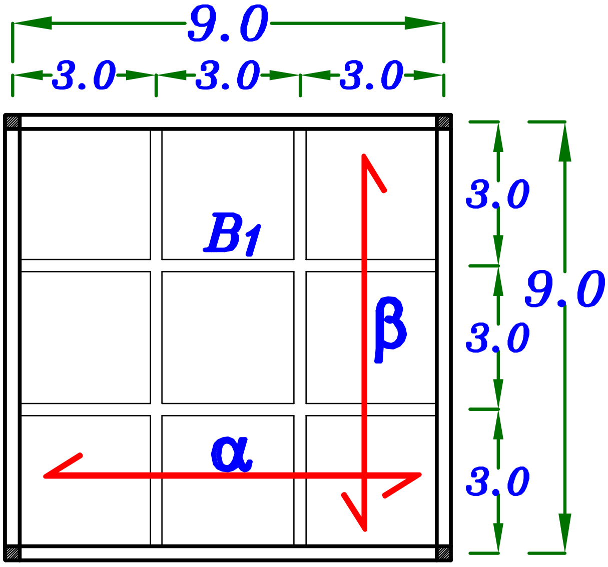
c – Calculate α , β By using Grashoff.



$$r = \frac{m\,L}{m'\,L_s} = \frac{(1.0)\,9.0}{(1.0)\,9.0} = 1.0$$

$$\alpha = \frac{r^4}{1+r^4} = \frac{(1.0)^4}{1+(1.0)^4} = 0.50$$

$$\beta = \frac{1}{1+r^4} = \frac{1}{1+(1.0)^4} = 0.5$$

e – Calculate the reduction Factor of the B.M. $\left(\frac{\sin\theta}{\sin 90}\right)$

$$X = 3.0\ m \quad , \quad \frac{L}{2} = 4.5\ m$$

$$\theta_{B_1} = \frac{3.0}{4.5} * 90^\circ = 60^\circ$$

$$M_{des.} = \frac{w * L^2}{8} * \frac{\sin\theta^\circ}{\sin 90^\circ} = M_{U.L.}$$

$$\frac{w * 9^2}{8} * \frac{\sin 60^\circ}{\sin 90^\circ} = 196.76\ kN.m. \longrightarrow w = 22.44\ kN/m$$

$$w = 22.44 = w_{av.} * \alpha * \alpha = w_{av.} * 3.0 * 0.5 \longrightarrow w_{av.} = 14.96\ kN/m$$

$$w_{av.} = w_s + \frac{\text{Total Weight of Panelled Beams}}{L * L}$$

$$w_{av.} = \left[1.4(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)\right] + 1.4\left[\frac{b\,(t-t_s)\,[2*L + 2*L_s] * \gamma_c}{L * L_s}\right]$$

$$14.96 = \left[1.4\,(0.1*25 + 1.5) + 1.6\,(L.L.)\right] + 1.4\left[\frac{0.25\,(0.7-0.1)\,[2*9.0 + 2*9.0] * 25}{9.0 * 9.0}\right]$$

$$\therefore \boxed{L.L. = 4.39\ kN\backslash m^2}$$

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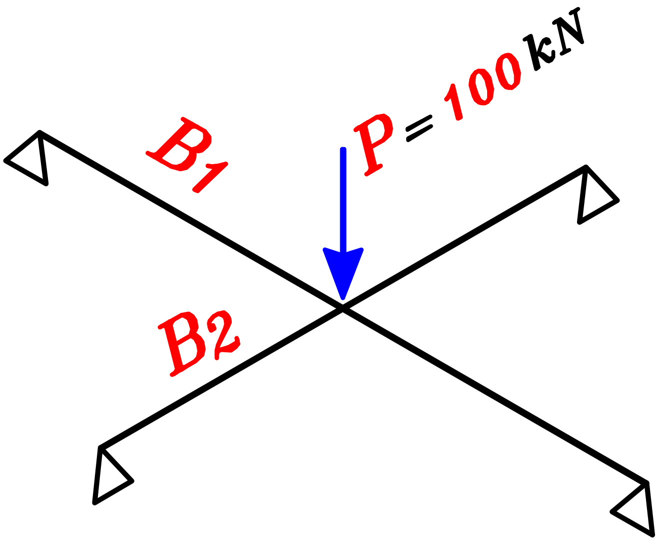
Example.

B_1 $(250*600)$

$L_1 = 7.0\ m$

B_2 $(300*700)$

$L_2 = 5.50\ m$



Deflection of beams B_1, B_2

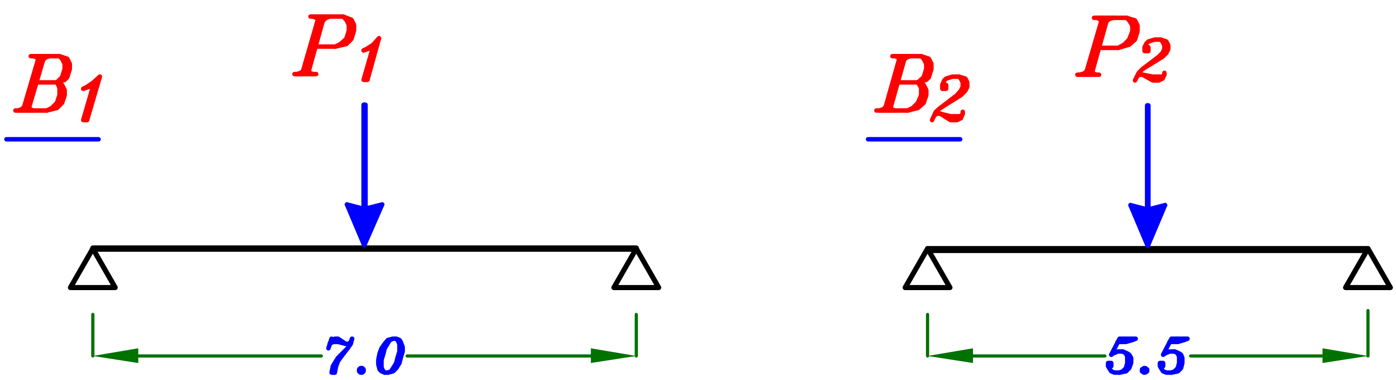
$$\boxed{\delta = \frac{P L^3}{48 E I}}$$

Reruired.

Find the amount of concentrated load will act on each beam.

Solution.

الحمل P سيتوزع على الكمرتين B_1 & B_2



$$\therefore P_1 + P_2 = P = 100\ kN$$

$$\boxed{P_1 = 100 - P_2} \quad \text{------} \quad (1)$$

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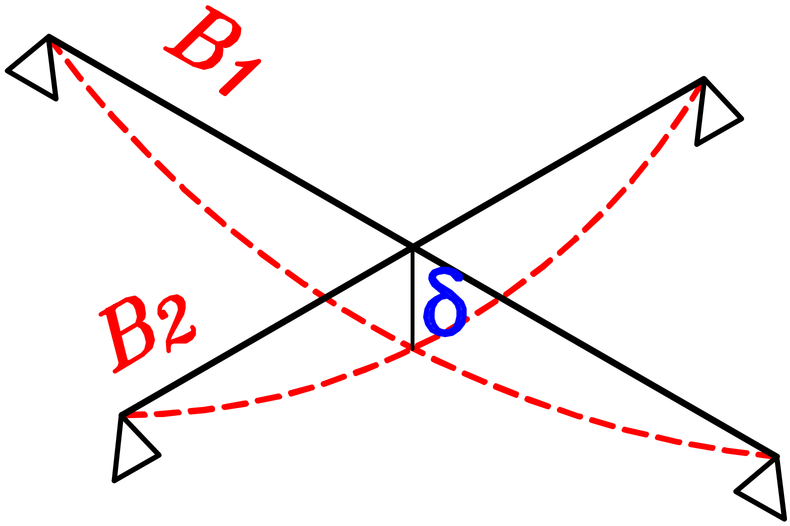
$$B_1\ (250 * 600) \longrightarrow I_1 = \frac{0.25 * 0.60^3}{12} = 4.5 * 10^{-3}$$

$$\delta_1 = \frac{P_1 L_1^3}{48 E I_1} = \frac{P_1 (7.0)^3}{48 E (4.5 * 10^{-3})} = 1587.9 \frac{P_1}{E}$$

$$B_2\ (300 * 700) \longrightarrow I_2 = \frac{0.30 * 0.70^3}{12} = 8.57 * 10^{-3}$$

$$\delta_2 = \frac{P_2 L_2^3}{48 E I_2} = \frac{P_2 (5.5)^3}{48 E (8.57 * 10^{-3})} = 404.4 \frac{P_2}{E}$$



$$\because \delta_1 = \delta_2 = \delta$$

$$\therefore \boxed{1587.9 \frac{P_1}{E} = 404.4 \frac{P_2}{E}} \quad \text{-------} \quad ②$$

From ① & ②

$$\boxed{P_1 = 20.3 \text{ kN}} \quad \boxed{P_2 = 79.7 \text{ kN}}$$

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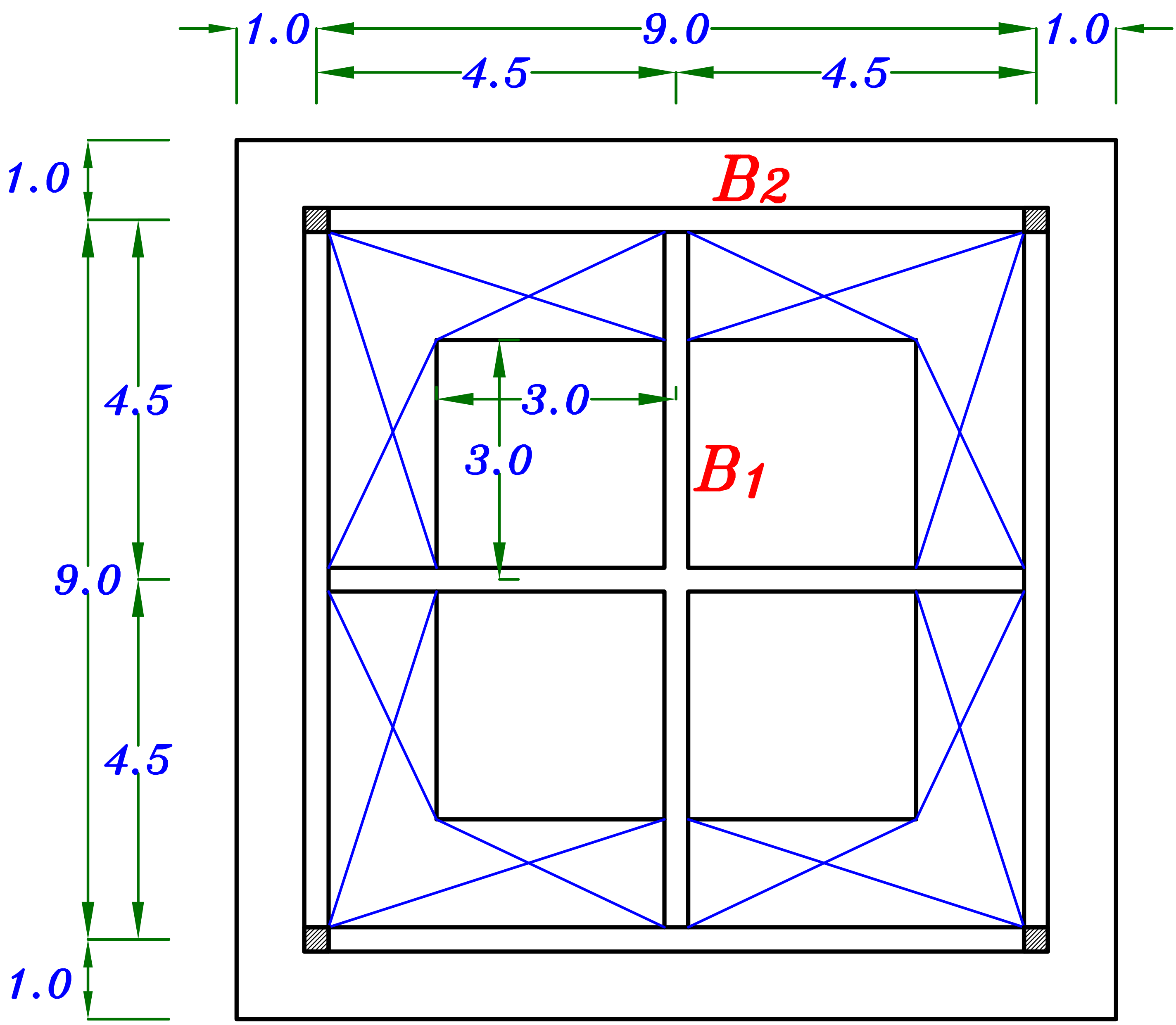
Example.



1 – Design the slabs & draw details of RFT. in plan.

2 – Design Beam B_1 & draw RFT. in elevation.

3 – Design Beam B_2 & draw RFT. in elevation.

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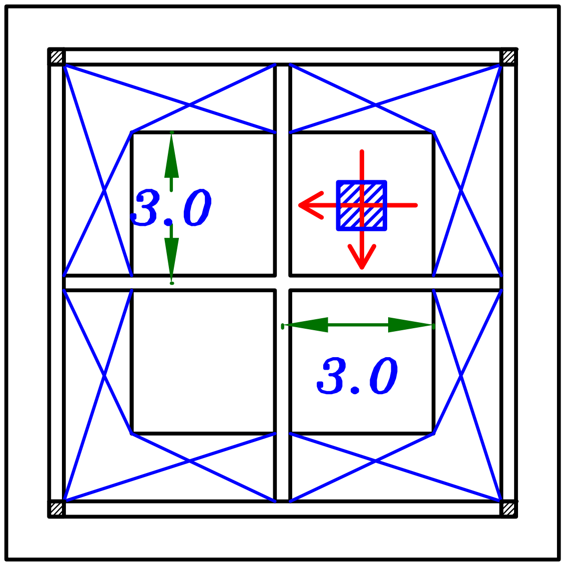
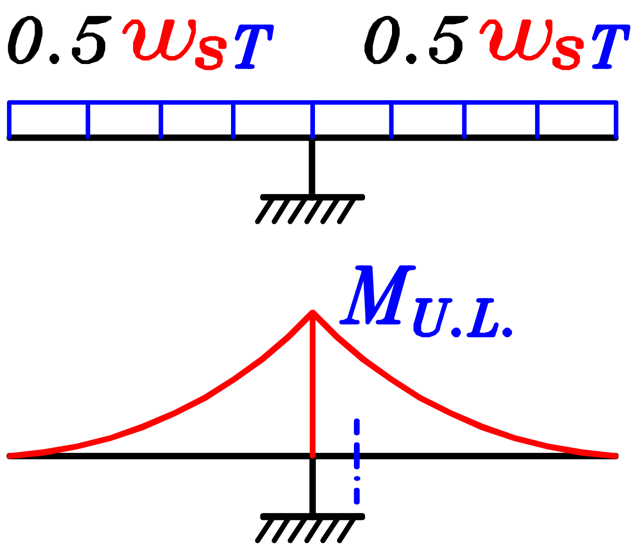
two cantilevers يتوزع حمل البلاطه على ال

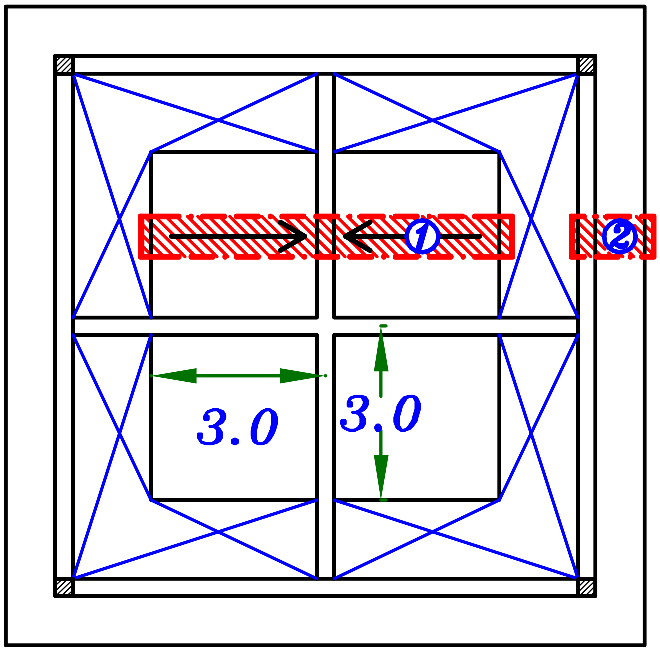
$0.5\,w_s$ أى يتوزع الحمل فى الاتجاهين كل اتجاه



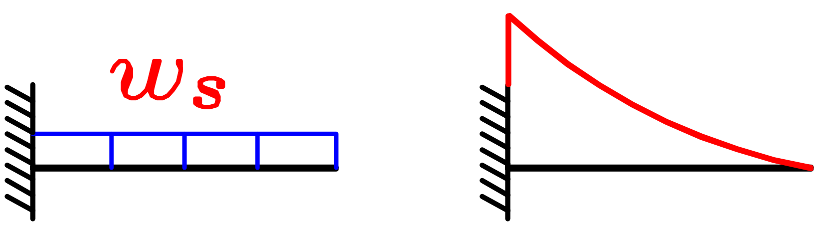
Slabs.

Strip ①





Strip ②



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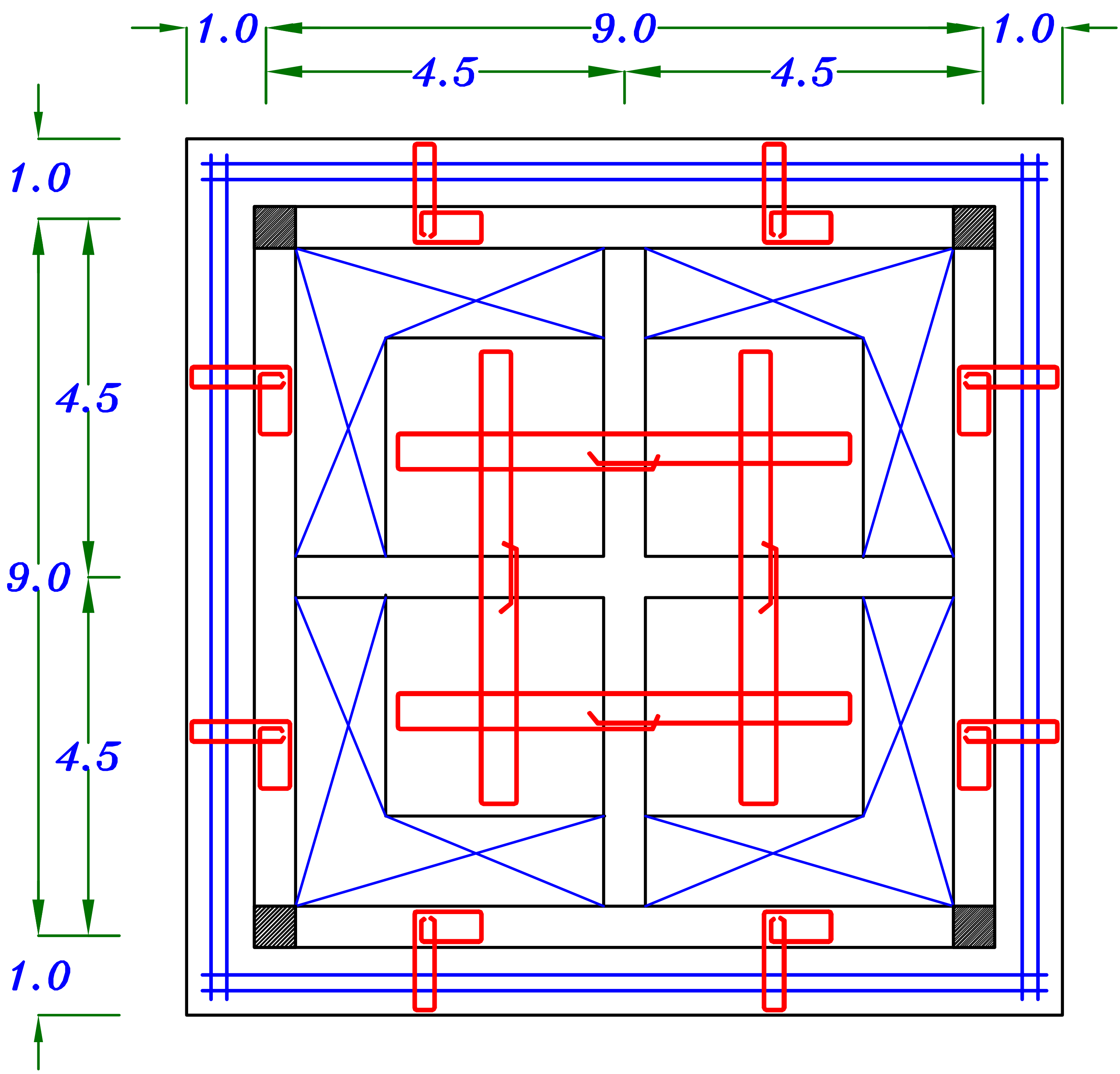
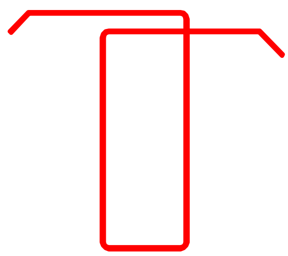
RFT. of Slabs.





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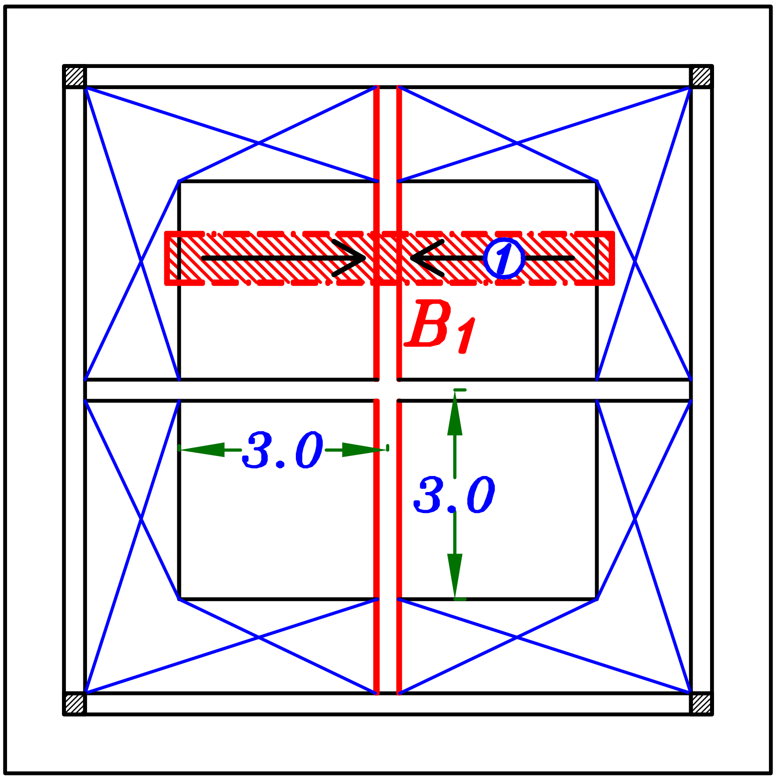
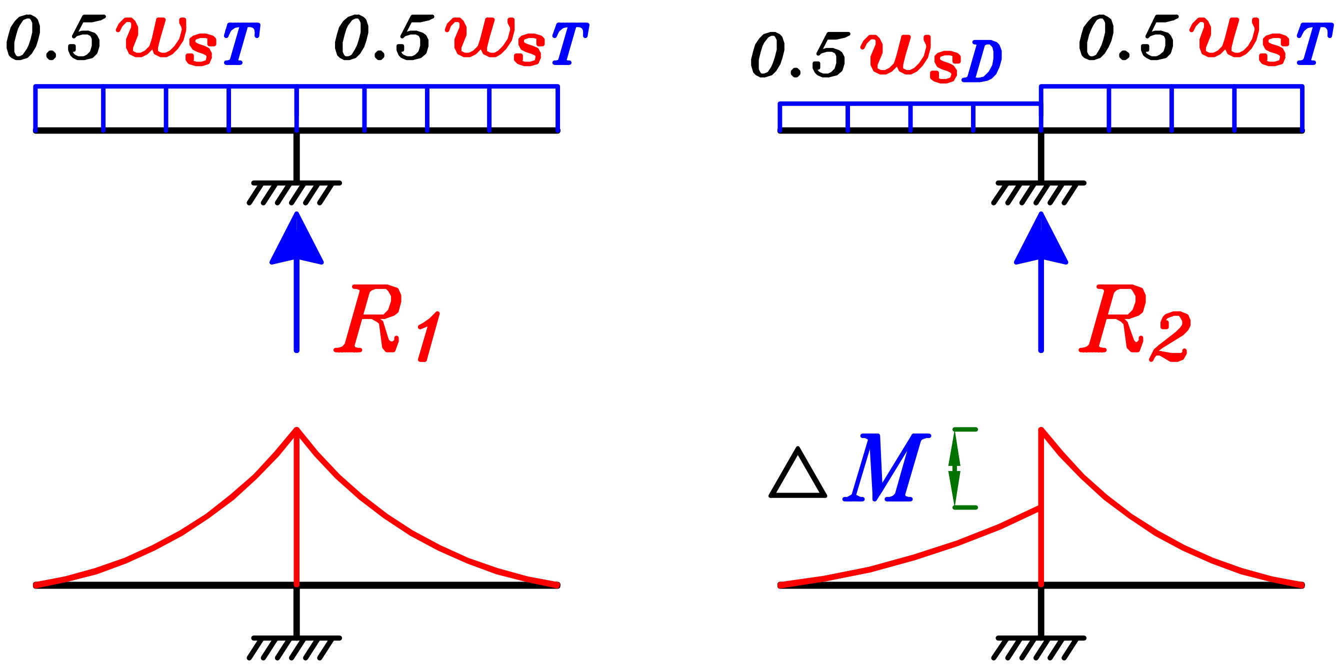
Panelled Beams B_1



Strip ①



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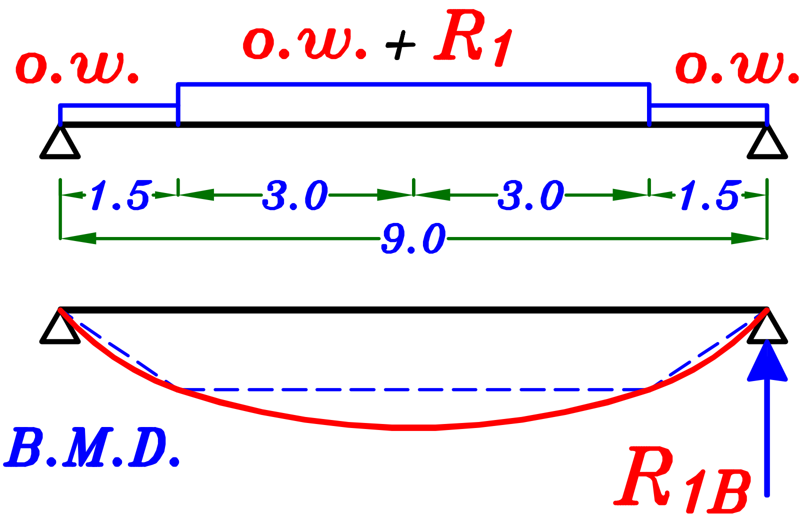

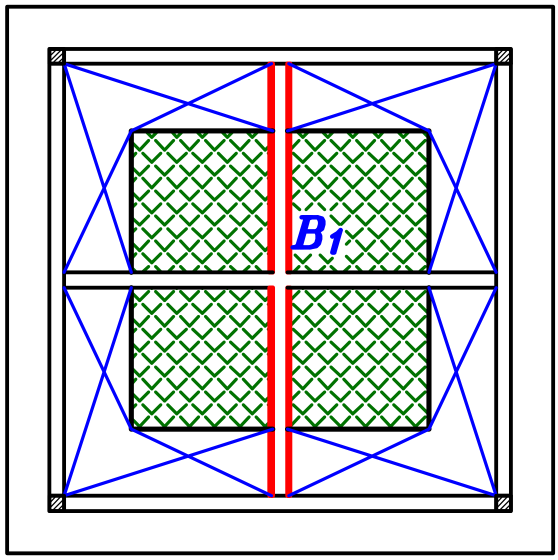

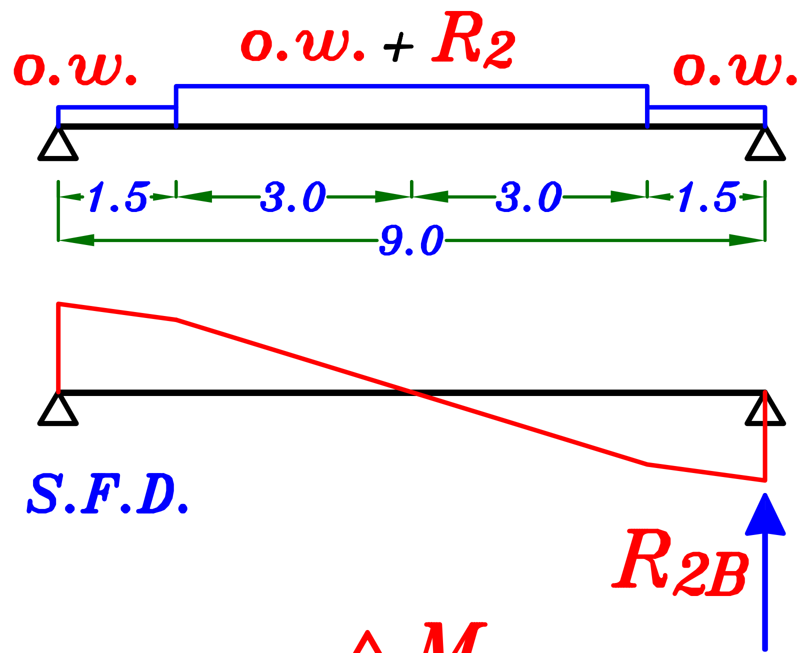

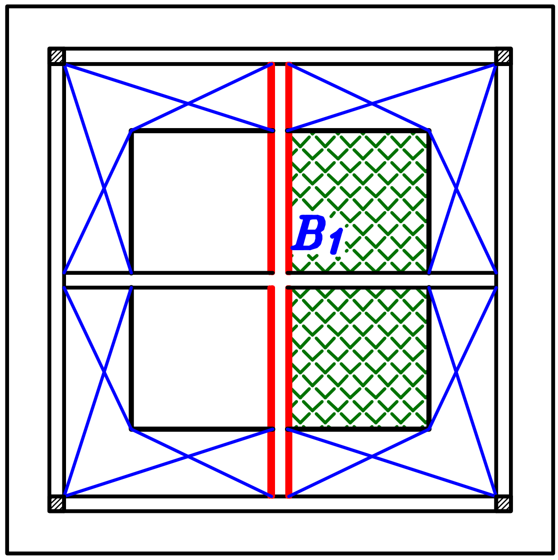

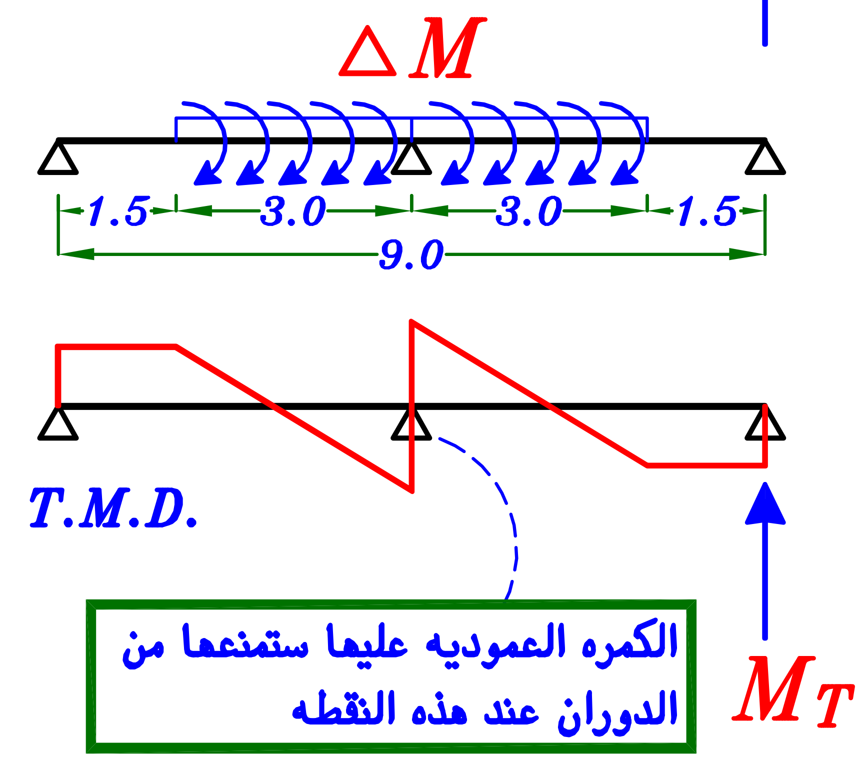

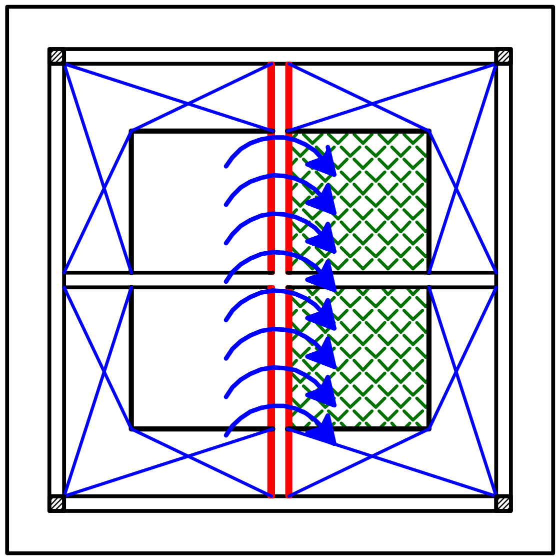

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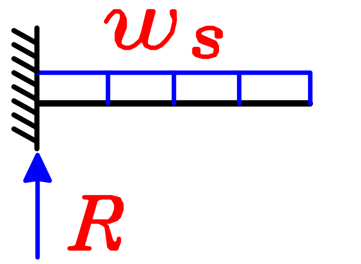
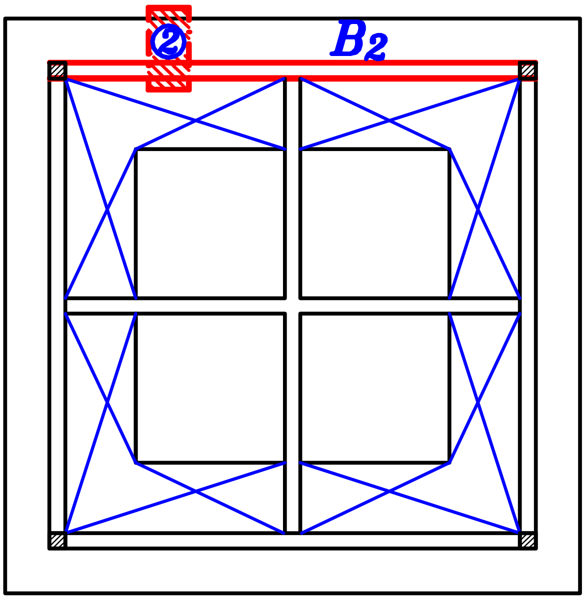
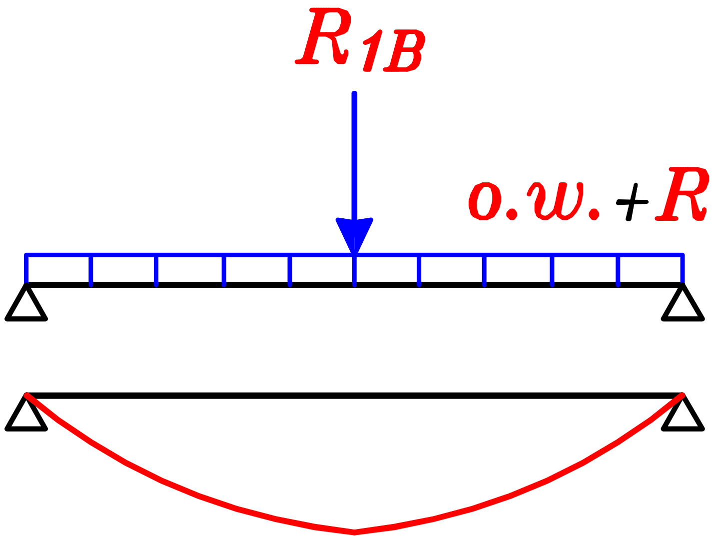
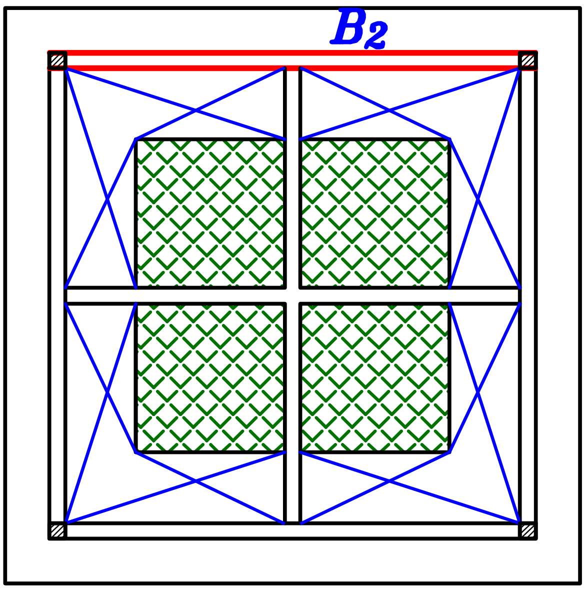
Edge Beam. B_2

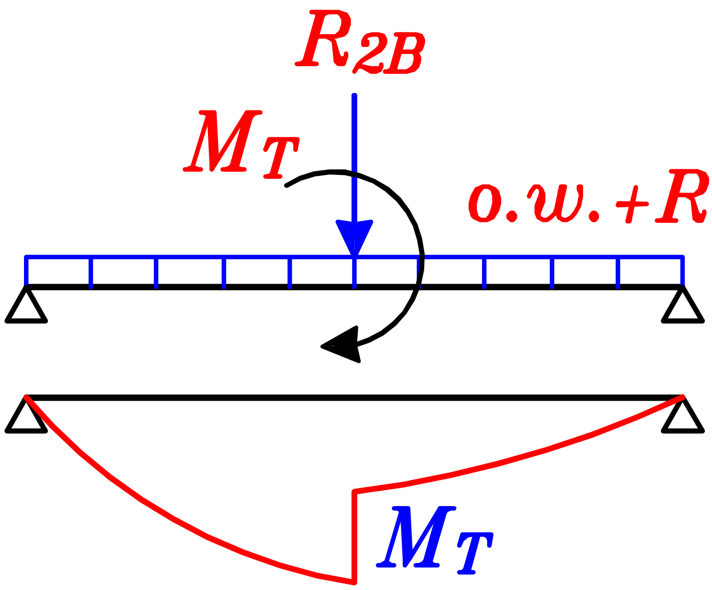
Strip ②

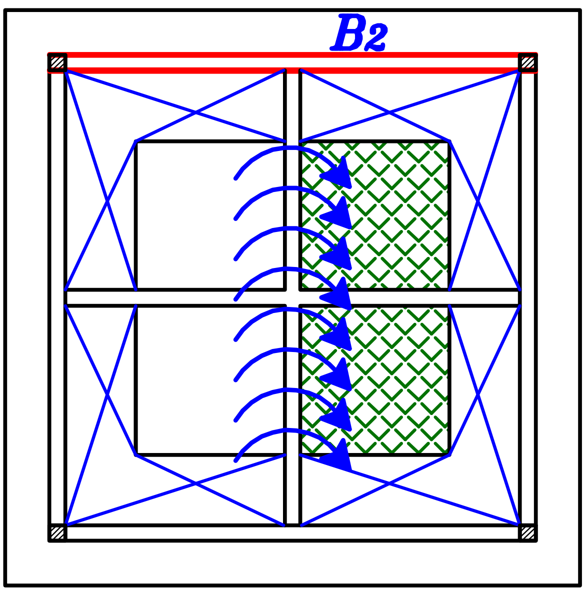












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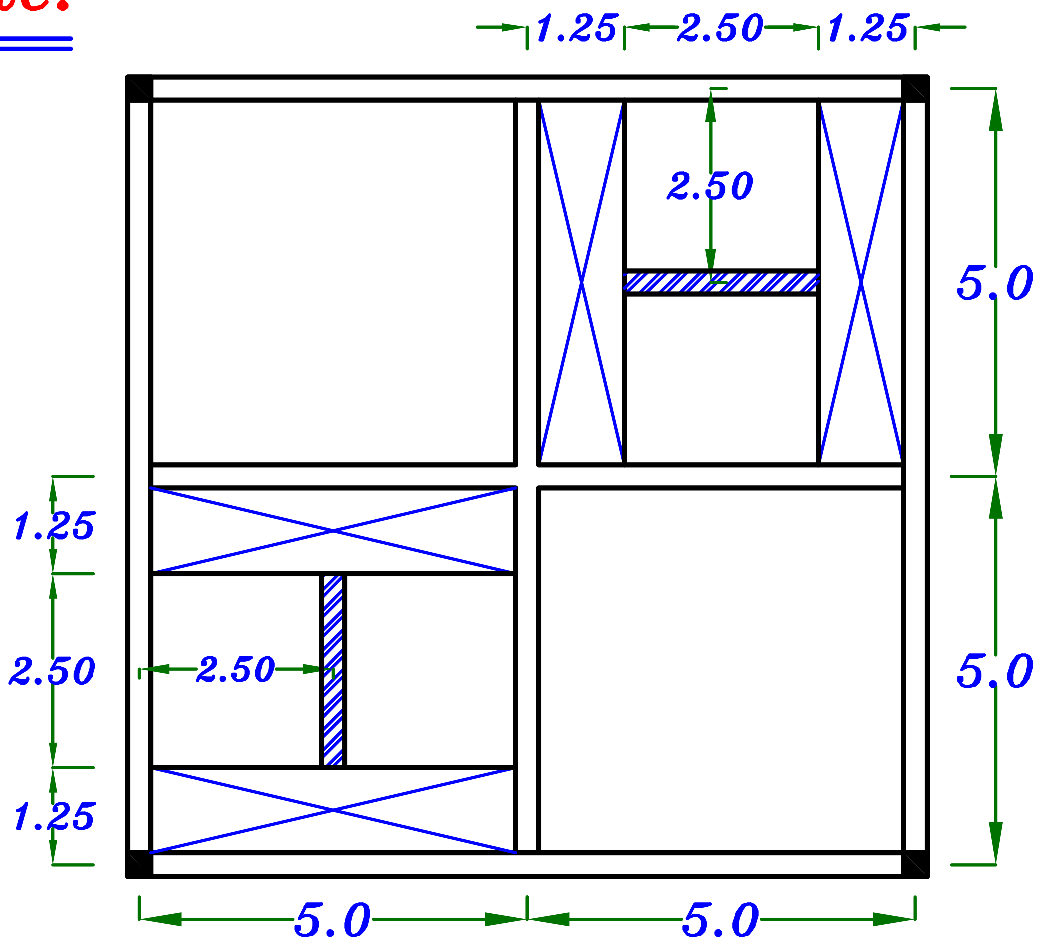
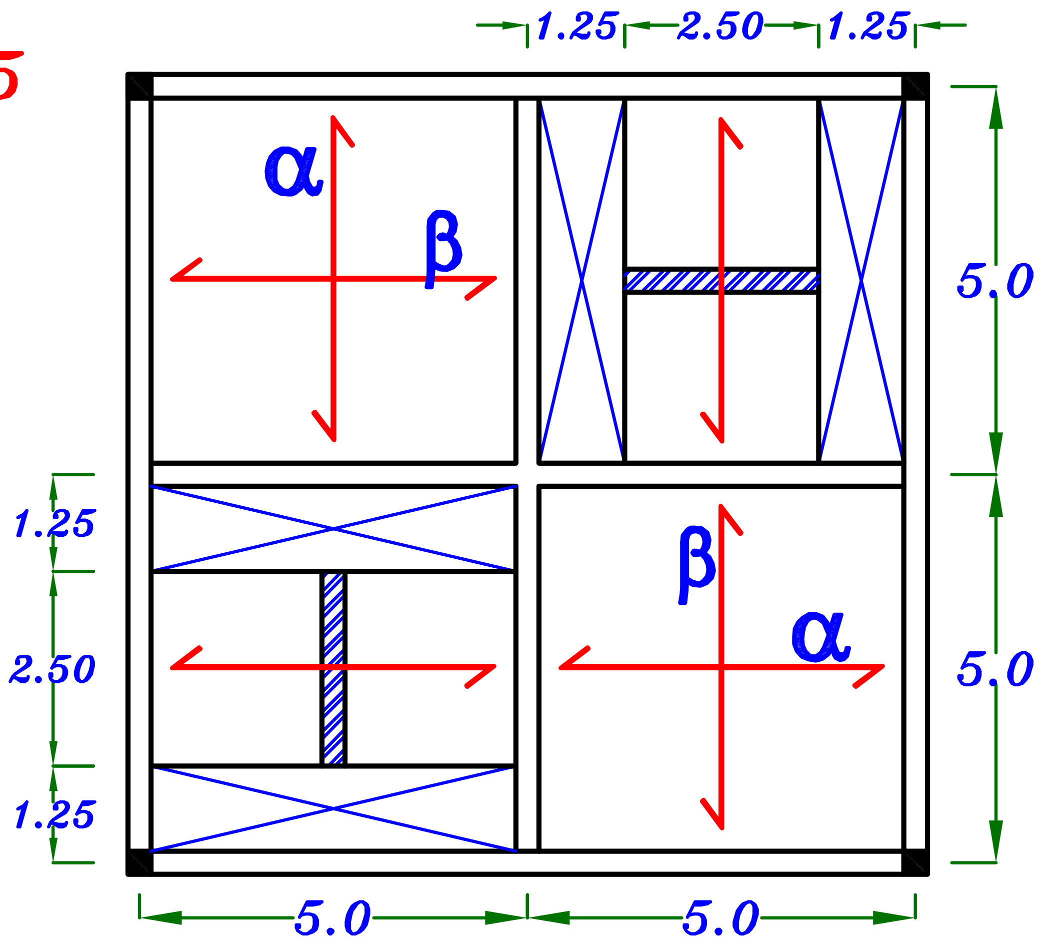
Example.



$\alpha = \beta = 0.35$



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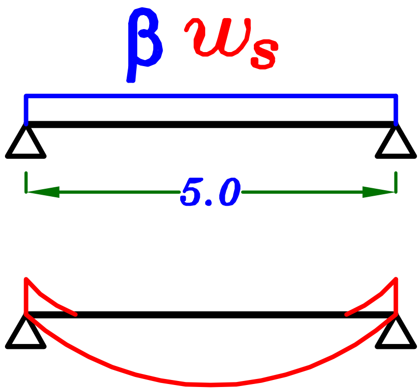
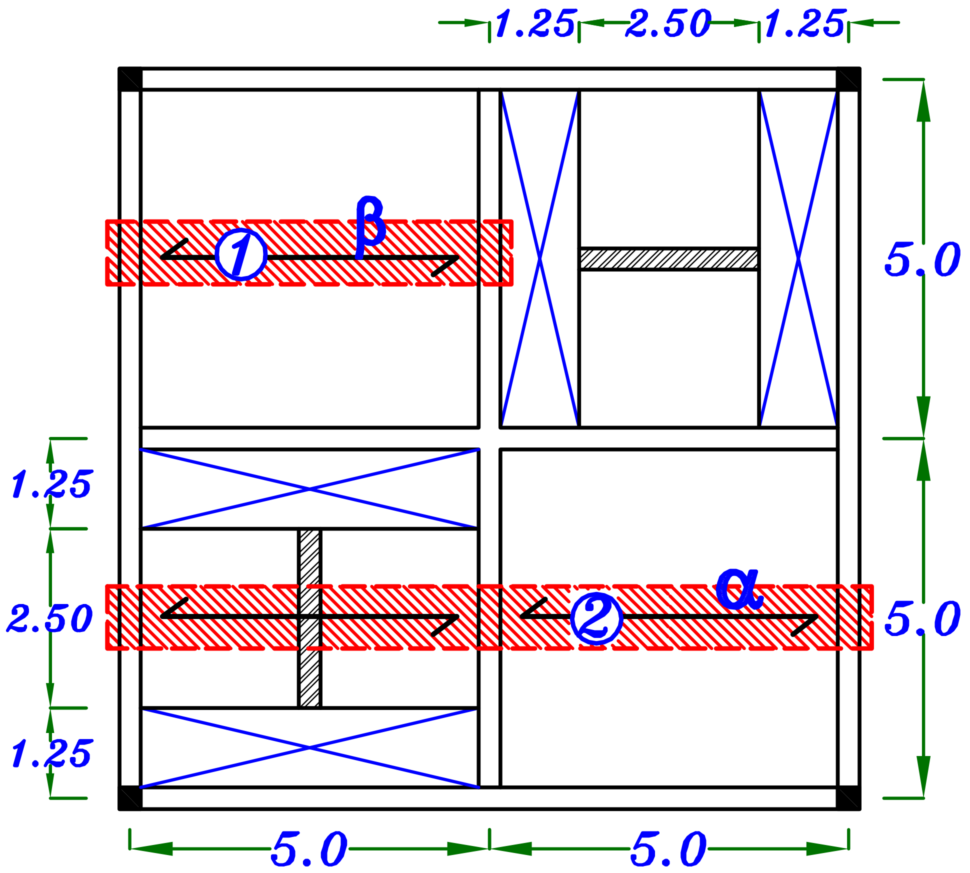
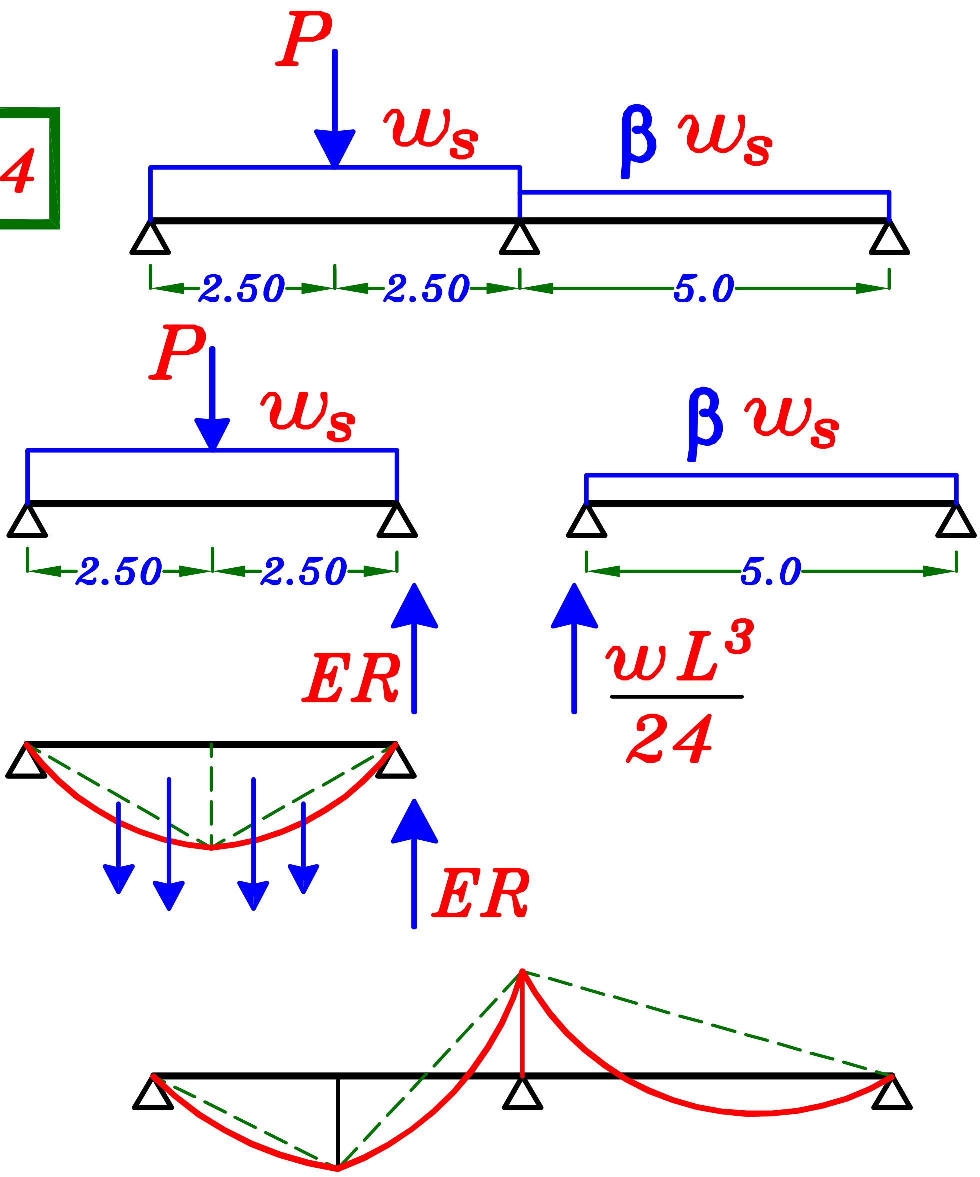
Strip ①





Strip ②

$$\boxed{P = b\,H\,\gamma_w * 1.4}$$



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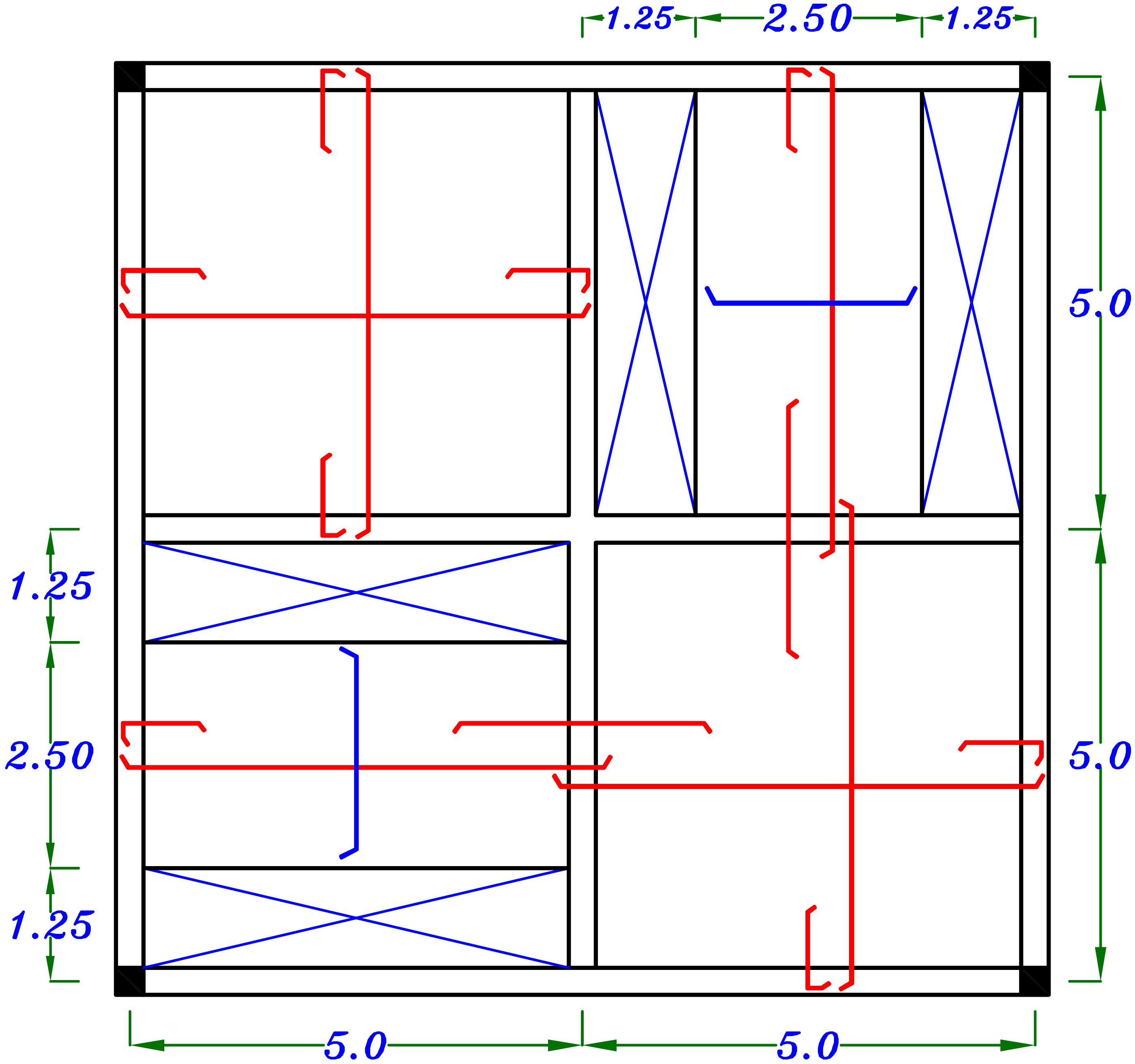
RFT. of the slab.



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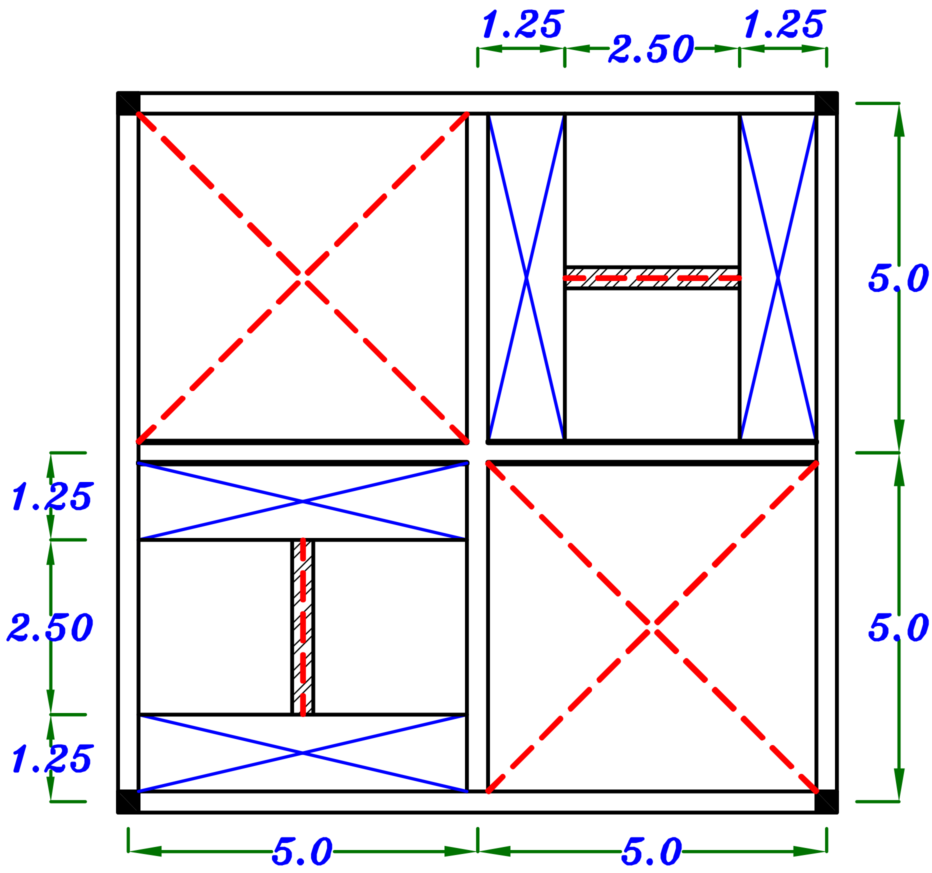
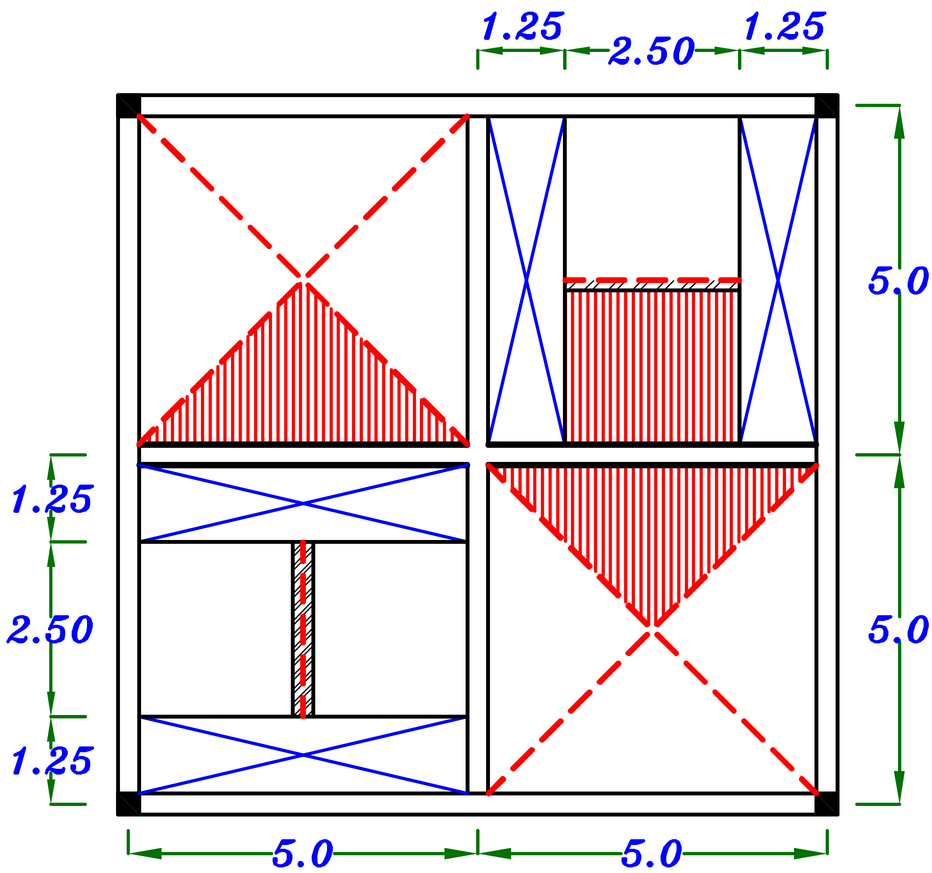
Panelled Beam.

Load Distribution لان الكمرتين فى المنتصف فمن الممكن حساب الاحمال عن طريق





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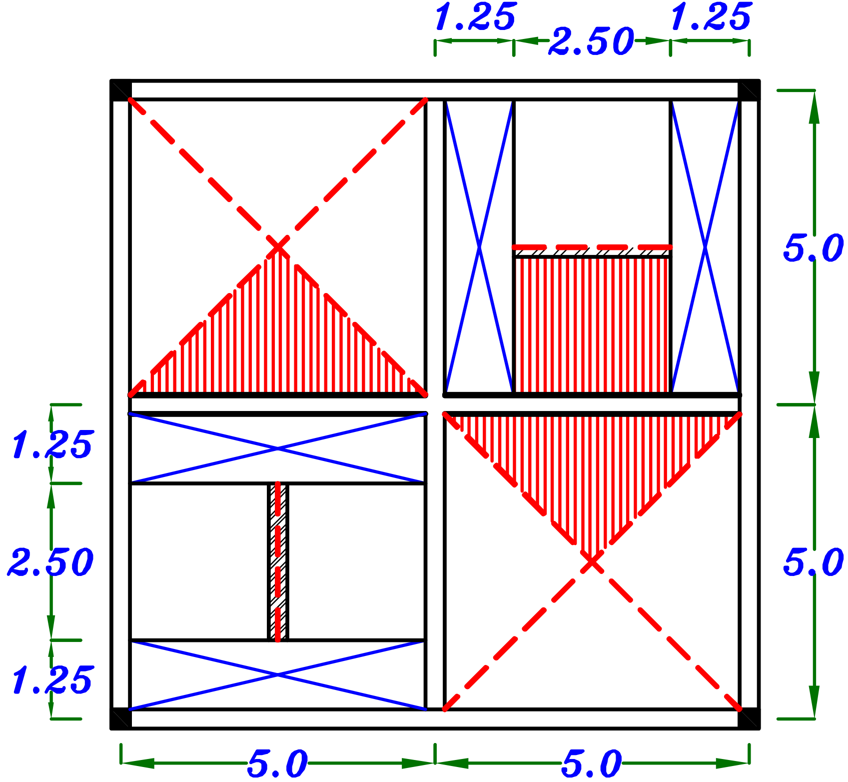
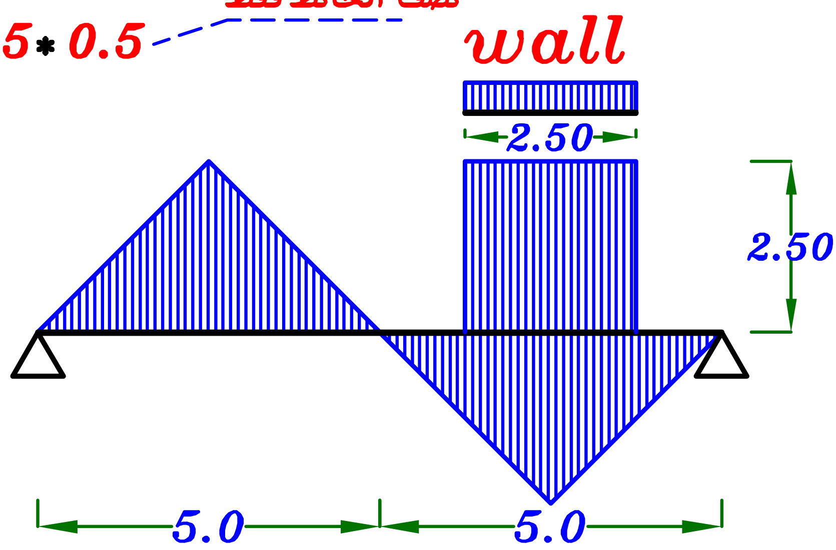
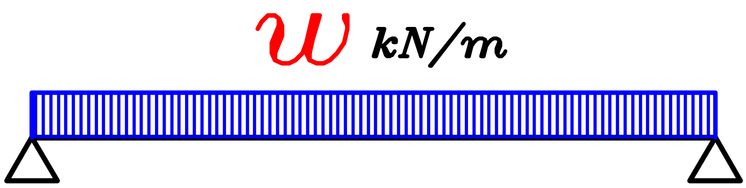
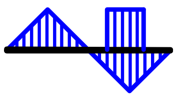


$$wall = (b H \gamma_w * 1.4) * 2.5 * 0.5$$

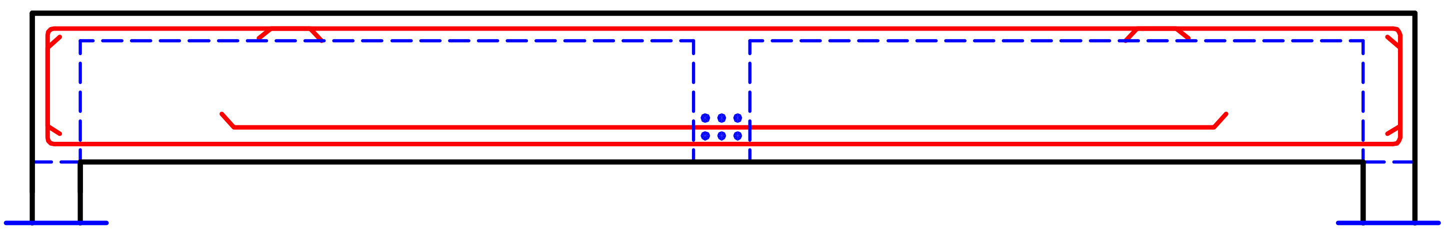
نصف الحائط فقط







$$w = o.w. + \sum \frac{area}{span} * w_s + \sum \frac{wall\ weight}{span}$$



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